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THE INFLUENCE OF STRAIN ON THE SHEAR STRENGTH PARAMETERS OF A HIGHLY PLASTIC REMOULDED, HOMIONIC CLAY SOIL

by

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The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance a thesis entitled "THE INFLUENCE OF STRAIN ON THE SHEAR STRENGTH PARAMETERS OF A HIGHLY PLASTIC, REMOULDED, HOMIONIC CLAY SOIL", submitted by ALDEN ERNIE DAHLMAN in partial fulfilment of the requirements for the degree of Master of Science.

ABSTRACT

The problem of this thesis was to investigate the influence of strain on the shear strength parameters of calcium, magnesium, potassium and sodium modifications of a normally consolidated, remoulded, lacustrine clay using the CFS (Cohesion-Friction-Strain) triaxial test technique. Three tests at consolidation pressures of 2, 4 and 6 kg/cm^2 were performed on each modification and on the natural soil.

For the soils of this program cohesion was shown to peak at relatively low strains, to be a function of consolidation pressure, and to be relatively unaffected by the physico-chemical characteristics of the system. It was postulated that the peak cohesion is a measure of the net repulsive forces that arise due to the external confining pressure and that it may be considered to reflect the inertia of the system to imposed shear strains.

Mobilized frictional resistance was shown to reach its maximum value at relatively large strains, to be related to the type of adsorbed cation, and to be relatively unaffected by the consolidation pressure. It was postulated that the mobilized frictional resistance is a manifestation of interparticle interference and the reorganization of electrostatic forces. This postulate appeared to allow a continuity of strength concepts through "pure" sand to "pure" cohesive soil. It is suggested that the nature of frictional resistance of a soil is primarily determined by the type of diffuse double layers present

and that these layers are thin for high angles of internal friction (K - 18.6°, Mg - 13.6°, Rem. - 13.4°, Ca - 12.0°) and thick for low angles (Na - 2.3°). These angles are only applicable to the soil of this program and the CFS test technique.

It is concluded that the defined cohesion and friction components are so closely interrelated that the definition of two separate components is unrealistic from a practical viewpoint. The CFS test shear strength components do, however, appear to be mechanically independent and thus are useful in research.

It is recommended that future research investigate the relation of cohesion and friction to plasticity characteristics and the strength components of over consolidated soils using the CFS test technique.

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GLOSSARY OF TERMS AND SYMBOLS

TERMS

Adsorbed Water -*Water in a soil mass, held by physico-chemical forces, having physical properties substantially different from adsorbed water, or chemically combined water, at the same temperature and pressure.

Absorbed Water -*Water held mechanically in a soil mass and having physical properties not substantially different from ordinary water at the same temperature and pressure (Pore Water).

Angle of Obliquity - *The angle between the direction of the resultant stress or force acting on a given plane and the normal to that plane.

Bonding Energies - Energies that are capable of bonding charged elements to a particle.

CEC - Cation Exchange Capacity - the quantity of cations adsorbed by the clay mineral in me/100 gms a.d.s. as determined by the titration procedure.

CFS - Cohesion - Friction - Strain - used in this thesis in reference to the triaxial test technique employed.

Deviator Stress -*The difference between the major and minor principal stresses in a triaxial test.

Double Layer - The double layer is composed of an internal monolayer and an outer diffuse layer.

Effective Stress -*The average normal force per unit area transmitted from "grain" to "grain" of a soil mass. It is the stress that is effective in mobilizing internal friction.

Homionic - When all the exchange positions of a clay mineral are occupied by one type of cation.

Macroscopic - Visible to the naked eye.

Major Principal Stress -*The largest (with regard to sign) principal stress.

me/100 gms. a.d.s. - Milliequivalents per 100 grams of air dried soil. A milliequivalent is the amount of a reagent required to combine and react with 1/1000th of an atomic weight of hydrogen.

Minor Principal Stress - *The smallest (with regard to sign) principal stress.

Normally Consolidated Soil Deposit - *A soil deposit that has never been subjected to an effective pressure greater than the existing overburden pressure and that is also completely consolidated by the existing overburden pressure.

Overconsolidated Soil Deposit - *A soil deposit that has been subjected to an effective pressure greater than the present overburden pressure.

Polarizability - The ability to deform charges from spherical symmetry to give rise to polar action.

Pore Water - All the free water in the clay-water system; i.e., excludes adsorbed water. (Absorbed water).

Salt Content - The concentration of salts in the pore water expressed in me/100 gms. a.d.s. Taken as being equal to the numerical difference between total cations present and the CEC.

Shear Strength - *The maximum resistance of a soil to shearing stresses.

Shear Stress - *The stress component tangential to a given plane.

Specific Surface - Area of a material per unit mass, e.g., cm^2/gm.

Strain - *The change in length per unit of length in a given direction.

Total Cations Present - The total quantity of cations in me/100 gms. a.d.s. as determined by the flame photometer and includes both those cations adsorbed and those present in the pore water.

Zeta Potential - The difference between the potential of the immovable layer attached to the surface of the solid phase and the potential of the free water.

* "Glossary of Terms and Definitions in Soil Mechanics", ASCE Proceedings, Soil Mechanics and Foundations Division, SM4, October 1958.

SYMBOLS

A	Electrical attraction between particles
C'	Effective cohesion (Mohr-Coulomb failure criterion)
C_e	Effective cohesion (Coulomb-Hvorslev failure criterion)
C_ϵ	Cohesion at strain ϵ (CFS test definition on page 11)
e	Void ratio
G_s	Specific gravity of soil mass; includes all salts present
R	Electrical repulsion between particles
t_{90}	Time required to reach theoretical 90% consolidation as determined by the graphical construction from the square root of time fitting method.
t_{100}	Time required to reach theoretical 100% consolidation as determined by the graphical construction from the logarithm of time fitting method
u	Pore Water Pressure
β	Angle of Mohr failure envelope from Mohr-Coulomb analysis of CFS test data when $\bar{\sigma}_1 \sim \sigma_c$
σ_1	Total major principal stress
$\bar{\sigma}_1$	Effective major principal stress
σ_3	Total minor principal stress
$\bar{\sigma}_3$	Effective minor principal stress
$\sigma_d =$	$\sigma_1 - \sigma_3$ Deviator stress
σ_f	Total stress normal to failure surface
σ_p'	Effective prestress or preconsolidation pressure (Krey-Tiedemann analysis)

τ_f Shear stress at failure

ϕ' Effective angle of internal friction (Mohr-Coulomb failure criterion)

ϕ'_c, ϕ'_r Angles of shear strength (Krey-Tiedemann analysis)

ϕ'_e Effective angle of internal friction (Coulomb-Hvorslev failure criterion)

ϕ'_ϵ Angle of internal friction at strain ϵ (CFS test definition on page 11)

CHAPTER I

INTRODUCTION

1.1 The continuing search for explanations of the complex behavior of fine grained soils has led the soils engineer into the study of disciplines formerly considered beyond the scope of normal applied soil mechanics. Examination of clay soils on microscopic and molecular scales has brought into play the fields of clay mineralogy, physical chemistry and soil physics. Through these disciplines, the distinctive physical properties of fine grained soils such as electrical instability and high specific surface* combined with the chemical implications of ion exchange phenomena have been the basis for explanations of the engineering behavior of these soils at least in a qualitative sense. Of particular interest to the soils engineer are the structural units which are characteristic of the various clay minerals and their associated cation exchange phenomena.

1.2 The engineering properties of a given fine grained soil have been shown to vary with the nature of the exchangeable cation complex (Thomson, 1960; Hamilton, 1961; Thomson, 1963; Locker, 1963)**. The variations shown were largely qualitative in nature but because relationships do exist, research utilizing physico-chemical phenomena should continue. When quantitative expressions are derived, it seems reasonable to expect that an analysis of the engineering properties of a soil in the light of physico-chemical concepts would reveal a much more comprehensive picture of field behavior. Alternately, one might be able to tailor a soil on the basis of an analysis to yield the desired properties to suit

* See Glossary of Terms and Symbols
** References listed alphabetically in "List of References"

the field situation.

1.3 Previous research at the University of Alberta revealed that the nature of the adsorbed cation complex affected, to a considerable degree, the geotechnical properties, including shear strength, of a given clay soil. Investigations by P.A. Thomson (1960) and A. B. Hamilton (1961) illustrated the requirement of homionic modifications for a meaningful study of physico-chemical phenomena and showed that the type of adsorbed cation markedly affected the Atterberg Limits and consolidation characteristics of a clay-shale. The effect of adsorbed cations and pore water salt content on the shear strength of a normally consolidated, remoulded, highly plastic clay were studied by S. Thomson (1963) and J. G. Locker (1963). The extensive program carried out by S. Thomson on clay having a high proportion of the exchange complex occupied by a single cation species, notably calcium, magnesium, potassium or sodium, and various pore water salt contents indicated that strength variation could be attributed to these factors. Using consolidated undrained tests with pore pressure measurements, the sodium modifications yielded the lowest strengths. A decrease in salt content in the pore water caused a decrease in shear strength for all modifications. Grain size distributions and specific gravity determinations also varied with the adsorbed cation and salt concentrations. The program of J. G. Locker, based on Thomson's work, consisted of shear strength determination of homionic calcium and sodium clay modifications using controlled concentrations of salts in the pore water. Trends were established which substantiated earlier

work. Both workers adopted maximum principal stress ratio as their failure criterion and defined mohr envelopes on this basis. Maximum deviator stress occurred at essentially the same strain as maximum stress ratio and thus the criterion used was not critical. A series of three tests on a given modification were used to obtain an effective cohesion and friction angle value from the Mohr plot. The values thus obtained were used for a physico-chemical interpretation and comparison of soil strengths. Whereas friction angles could be interpreted and logically compared on this basis, cohesion intercepts were largely ignored as no definite trends were apparent.

1.4 All strength tests previously performed on essentially homionic modifications were consolidated undrained tests with pore pressure measurements. Recently, J. H. Schmertmann and J. O. Osterberg (1960) have developed a triaxial shear strength test termed the CFS test (Cohesion - Friction - Strain test). During this test at a constant rate of strain pore water pressure was varied to maintain a constant selected value of major principal effective stress ($\bar{\sigma}_1$). Data were obtained at this $\bar{\sigma}_1$, after which the pore pressure was changed to maintain a new constant value of $\bar{\sigma}_1$. The procedure of alternating between two chosen values of $\bar{\sigma}_1$ permitted the fitting of two stress-strain curves to the data which were then mathematically interpreted to obtain cohesion and friction as a function of strain. The apparent value of this test stems from the fact that only one specimen was required to obtain all necessary data for a strength evaluation. Cohesion was shown to develop very rapidly with strain and to exhibit

a "peaking" effect a low strain whereas friction tended to develop more slowly and reach a maximum at larger percentage strains. The cohesion and friction terms as defined by Schmertmann and Osterberg were not construed to be fundamental properties but did appear to be mechanically independent.

1.5 In an attempt to further illustrate the effects of physico-chemical phenomena on the shear strength parameters of a fine grained soil, the subject of this program is to trace the development of cohesion and friction with strain using the CFS method of triaxial testing on modifications of a remoulded lacustrine clay whose exchange complex is predominantly occupied by a single cation species. Calcium, magnesium, sodium and potassium modified specimens and the natural soil were tested. Three tests were carried out on each modification at consolidation pressures of 2, 4 and 6 kg/sq. cm. and at constant pore water salt concentrations. A test was also performed on one undisturbed clay specimen.

1.6 The soil of this program was similar to that used by S. Thomson (1963) and J. G. Locker (1963) and the principles of preparing the modifications were identical. The primary difference lies in the test technique. The procedure is of particular interest because of the necessary slow rate of axial strain and the volume change that occurs during the test. Consequently, it is examined quite closely. Because the program was limited to three tests on each modification, qualitative behavior and trends overshadow quantitative results. It has been assumed that the reader has a basic knowledge of clay mineralogy and soil physics. For more information in these fields, other available literature or a summary by S. Thomson (1963, Appendices B,C,D) is recommended.

1.7 In the following chapters, a brief review will be made of pertinent literature including work at the University of Alberta, sample preparation discussed fully and the test technique examined carefully. An estimation of the errors in the results will also be presented. A complete presentation and discussion of results in physico-chemical and contemporary testing terms is followed by conclusions arising from this program and recommendations for future research.

CHAPTER II

LITERATURE REVIEW

Concepts of Shearing Strength

2.1 A review of current literature on the shear strength of cohesive soils has revealed three general trends of thought with regard to the source and nature of shear strength and its laboratory test manifestation. The classical Mohr - Coulomb failure concept of shear strength in terms of friction and cohesion components is well known. Arising from this concept, the Coulomb - Hvorslev failure criterion in terms of effective stresses implied at least the mechanical independence of the two strength components and forms the basis for the technique of Schmertmann which is described herein. The Krey - Tiedemann analysis and a techique developed by Palmer et al (1957) also suggest the independence of friction and cohesion components and will be described further. Analyses on the basis of the Mohr - Coulomb failure concept are still prevalent, particularly in stability problems. The advantage of this concept lies in its relative simplicity and expediency with respect to both laboratory strength determination and application to practical problems. A second approach to the concept of shear strength is that of the physical chemist. Shear strength is described and explained on a microscopic and molecular scale in terms of the basic structure of the soil particles and the electrical interactions between

the solid and liquid phases of a soil system. Lambe (1960) presents a strength equation in terms of the electrical forces acting in this system but indicates that division of strength into two components is difficult because of the interrelation of all factors involved. Rosenqvist (1955, 1959), who also adopts the physico-chemical approach to shear strength, states that there appears to be no fundamental difference between cohesion and friction. Work performed at the University of Alberta by Thomson (1963) and Locker (1963) indicated that strength variations, illustrated by effective friction angles according to the Mohr - Coulomb failure concept, could be qualitatively explained in terms of physico-chemical phenomena but cohesion intercepts showed no definite trends. A third approach to the concept of shear strength is that proposed by Schmid based on energy considerations. No physical difference between cohesion and friction was postulated and the ability of the Mohr - Coulomb strength theory to furnish a unique statement of the conditions obtaining at failure was severely questioned. The remainder of this section consists of an expanded explanation of these general trends of thought as they pertain to the interpretation of the results presented herein.

Mohr - Coulomb Failure Criterion

2.2 Originally proposed by Coulomb and later generalized by Mohr, the Mohr - Coulomb failure criterion in accordance with Terzaghis' fundamental concept of effective stress may be written:

$$\tau_f = C' + (\sigma_f - u) \tan \emptyset'$$

where: τ_f = shear stress at failure

C' = effective cohesion

σ_f = total stress, normal to failure surface

u = pore water pressure on failure surface

ϕ' = effective angle of internal friction.

In the Mohr diagram (FIGURE II.1(a)), this equation describes the shear strength envelope, tangent to all possible stress circles representing incipient failure, as a straight line.

Krey - Tiedemann Analysis

2.3 An elaboration of the original Coulomb equation is the Krey - Tiedemann criterion of failure. (Tschebotarioff, 1951, page 155). Hvorslev (1960) rewrote the Krey - Tiedemann equation in terms of effective stresses in the form:

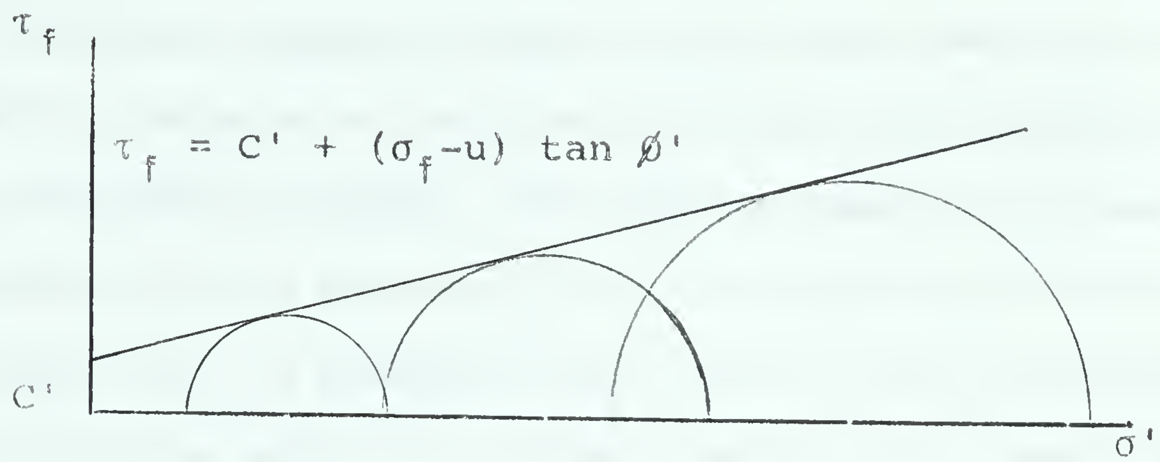
$\tau_f = \sigma p' \tan \phi' c + \sigma_f' \tan \phi'$

where: $C' = \sigma p' \tan \phi' c$

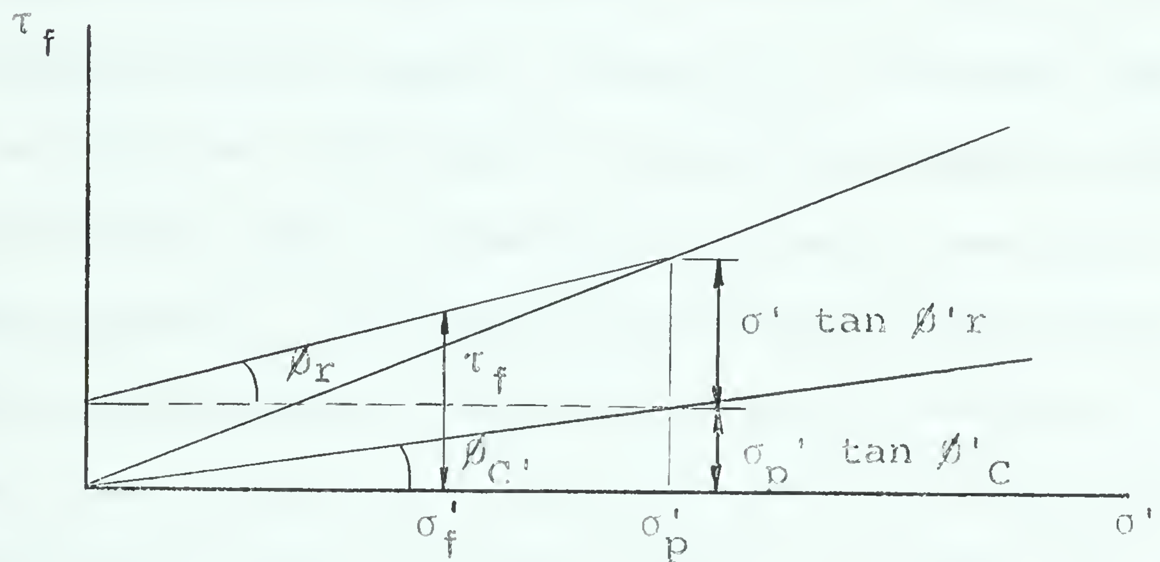
σ_p' = intial consolidation pressure.

The term, C' was interpreted by Hvorslev (1960) to be the cohesion intercept on the Mohr plot and the angles ϕ'_c and ϕ'_r were called angles of shear strength. The values C' and ϕ'_r were not considered to be the actual cohesion and angle of internal friction but merely mathematical components. This criterion did not consider the effect of sample expansion upon reduction of the maximum preconsolidation load but is important since it abandons Coulomb's concept of the cohesion having a constant value for a given material. The Krey - Tiedemann concept is illustrated in FIGURE II.1(b).

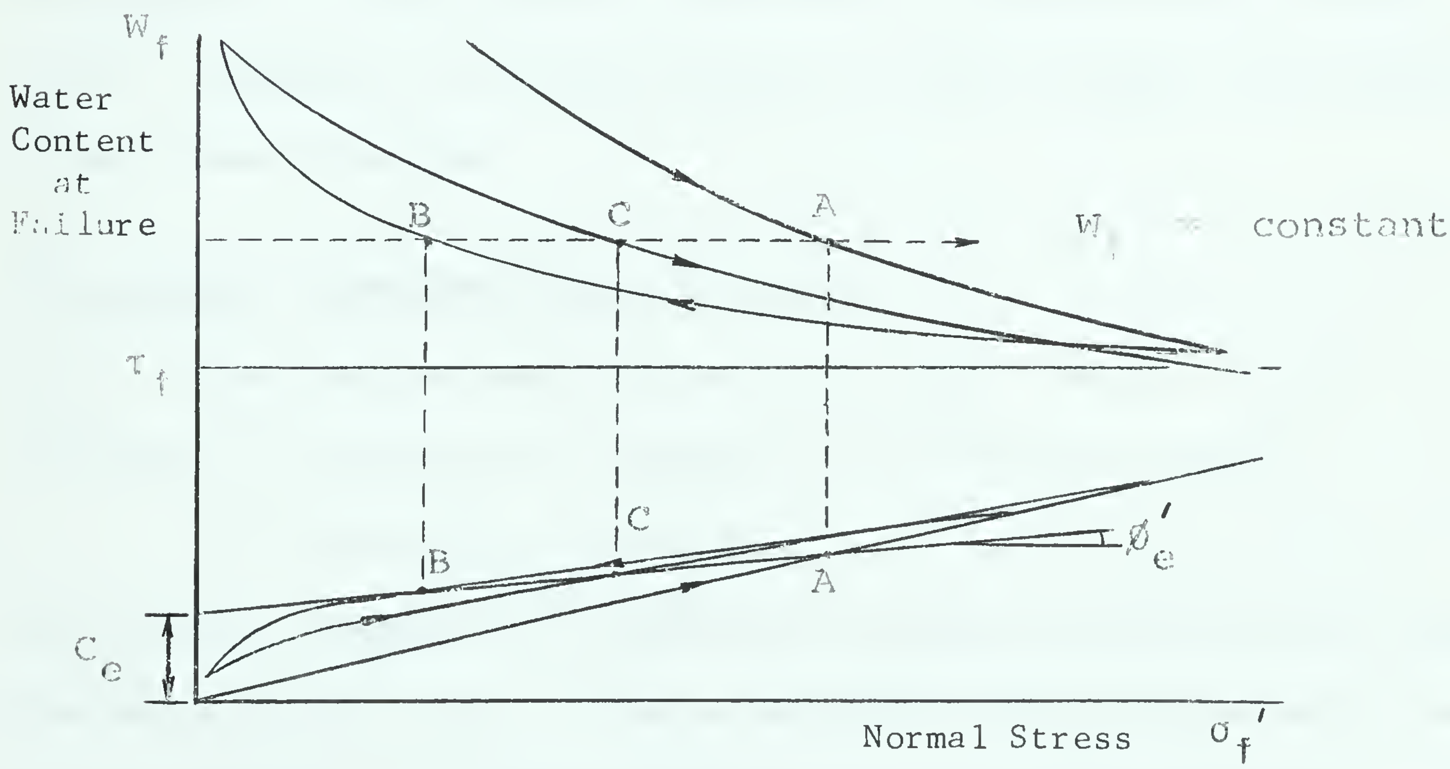
2.4 Palmer et al (1957) describe a method of separately evaluating friction and cohesion of soils using a consolidated direct shear test.



(a) Mohr Diagram with Mohr-Coulomb envelope



(b) Coulomb Shear Strength Diagram Illustrating Krey-Tiedemann failure Criterion (after Hvorslev, 1960)



(c) Separate Determination of Friction and Cohesion Components (after Hvorslev, 1960)

FIGURE II.1 Concepts of Shearing Strength

The frictional component of shear strength was treated as a transient quantity, dependent entirely upon an acting normal pressure in excess of the consolidation pressure. The cohesion component of shear strength was considered to be a permanently built in characteristic of the soil and dependent upon the pressure to which the soil had been consolidated. One set of test points for a shear strength versus normal pressure plot were obtained by consolidating the samples under a given normal load and shearing without removal or reduction of the normal loads. A second set of points were determined by consolidating companion samples under similar normal loads but shearing of the samples took place at very low normal pressures. The joining of the strength points derived from samples consolidated to identical pressures but sheared at different normal loads yielded a unit cohesion on the shear strength axis which represented the shear strength retained by the sample after consolidation and removal of normal loads. This method of evaluation was based on the Krey - Tiedemann total stress analysis of shear strength and completely ignored pore pressures.

The Coulomb - Hvorslev Failure Criterion

2.5 The Hvorslev modification of the Mohr - Coulomb failure criterion is illustrated in FIGURE II.1(c) and may be written:

$$\tau_f = Ce + (\sigma_f - u) \tan \emptyset'e.$$

The friction component is a function of effective normal stress on the failure plane, $\sigma_f' = \sigma_f - u$ and is defined by the shear strength line obtained when σ_f' is varied while the cohesion and rheological components remain constant. The angle of inclination, $\emptyset e'$ is called the

effective angle of internal friction and is generally a constant for a given soil. The effective cohesion component, C_e, is defined as the zero intercept on the Hvorslev strength plot and, in the case of fully saturated soils, is a direct function of the water content or void ratio. In this analysis, the assumption of equal cohesion at equal void ratio - also implies no significant difference in the geometric structure in a given series at the time of failure, i.e. at points A, B and C in FIGURE II.1(c). It should be noted that the effective cohesion, C_e, and the effective angle of internal friction, ϕ_e, were originally termed the "true" cohesion and "true" angle of internal friction. Literature prior to 1960 generally retained the latter terms in reference to the Coulomb - Hvorslev strength parameters.

The CFS Test

2.6 The laboratory test termed the Cohesion - Friction - Strain (CFS) test was briefly described in the preceding chapter (Paragraph 1.4). This test allows the determination of the strain - mobilization of the cohesion and friction components of the resistance of a soil to shear stress. The components were defined by Schmertmann and Osterberg (1960) in the following manner:

> Cohesion (C_ϵ) - "The cohesion of a soil, at any strain, is the shear stress developed on the plane of Mohr envelope tangency at that strain, if the intergranular stress on that plane could be reduced to zero without significant change in soil structure".
>
> Angle of Internal Friction (ϕ_ϵ) - "The angle of internal friction, at any strain, is the angle whose tangent is the ratio of the change in shear stress to the change in normal intergranular stress occurring on the plane of Mohr

> envelope tangency at that strain, during a stress change occurring without significant change in soil structure".

The expression "without significant change in soil structure" in these definitions implies that a change in void ratio of less than 1% occurs in conjuction with a change in $\bar{\sigma}_1$ (a "hop" to the adjacent constant $\bar{\sigma}_1$ curve) at the same strain. At any given strain, the mathematical expressions for the components, as derived from the Coulomb - Hvorslev failure equation, are:

$$\phi_\epsilon = \sin^{-1} \frac{\Delta\sigma_d}{2(\Delta\bar{\sigma}_1) - \Delta\sigma_d}$$

and: $$C_\epsilon = \frac{\sigma_d/2 - (\bar{\sigma}_1 - \sigma_d/2)\sin\phi_\epsilon}{\cos\phi_\epsilon}$$

where: σ_d = deviator stress = $(\bar{\sigma}_1 - \bar{\sigma}_3)$.

It should be noted that σ_d and $\bar{\sigma}_1$ in the expression for C_ϵ must be taken from the same curve, but can be taken from either one.

2.7 Schmertmann and Osterberg (1960) performed CFS tests on a variety of machine extruded and undisturbed, saturated, normally consolidated clay specimens. The samples tested had plasticity indices ranging from 7 to 56. This program yeilded the following observations and conclusions:

1. The Coulomb - Hvorslev failure criterion was shown to be valid at any strain.

2. The measured cohesion and friction strength components appeared to be mechanically independent, with the maximum cohesion generally developing at a very low strain while the friction component required a much greater strain to reach its maximum value. In some cases, the cohesion component demonstrated a pronouced "peaking

effect" after which the cohesion decreased as the friction component continued to increase.

3. The cohesion of a given clay at a fixed water content was not found to be a constant but depended on the soil structure. Structural differences were illustrated on a given type of clay by testing an undisturbed sample, natural remoulded specimens and samples remoulded with sodium phosphate compounds. Cohesion was found to vary at a given water content but all clays tested were found to exhibit about the same value of maximum cohesion at a given preconsolidation stress. Structural differences were responsible for different moisture contents under a fixed preconsolidation stress.

4. The maximum friction angles determined for the various soils tested did not vary with preconsolidation stress in a consistent manner.

2.8 The final techique of CFS - testing used in the program described in paragraph 2.7 permitted the determination of the components C_ϵ and $\emptyset_\epsilon$ from a test on a single specimen. The question of the validity of using a single specimen arose; that is, does a one-specimen test, which defines two stress-strain curves, yield the same computed results as would be obtained from two initially identical specimens each tested at constant $\bar{\sigma}_1$, and each defining only one of the stress-strain curves of the one-specimen test. Schmertmann (1962) compared one and two specimen CFS - tests on Ottawa sand, undisturbed cemented sand, sensitive clay, and six machine extruded soils with plasticity indices between 4 and 21% and

at 105% and arrived at the following conclusions:

1. Excellent qualitative agreement was obtained between the two methods in the curve of cohesion and friction against strain. Quantitative agreement on the maximum cohesion values was excellent.

2. A general tendency for cohesion to decrease, and friction to increase more rapidly with strain in the two-specimen tests was evident.

3. Excellent comparative one and two-specimen test agreement was shown by tests on the undisturbed samples.

2.9 Investigations of the microscopic behavior of soil-water systems under stress have led to numerous postulates as to the source and nature of shear strength in clay soils. Lambe's explanations in terms of interparticle forces (1960) and structure (1958) and the hypothesis of Rosenqvist (1955, 1959) in terms of polarizability of the adsorbed cations are notable as comprehensive presentations on the physico-chemical aspects of shear strength.

Lambe's Concept of Shear Strength

2.10 "A Mechanistic Picture of Shear Strength" by Lambe (1960) presents a theoretical consideration of the shear strength generation in fine grained soils utilizing forces between adjacent soil particles. The wet clay mineral was pictured as being surrounded by a double layer consisting of mobile ions in water. Environmental factors, such as temperature, concentration and type of ions in the double layer, and the dielectric properties of the double layer fluid were believed to

greatly influence the size and nature of the double layer and thus the arrangement of soil particles and the electrical forces acting between them. Lambe presented the following equation as applicable to a saturated, highly plastic, fine grained soil system:

$$\sigma' = \sigma - u - (R-A)$$

where: σ' = effective normal stress on the shear plane

σ = total normal stress on the shear plane

u = stress in the pore water

R = electrical repulsion between clay particles

A = electrical attractive forces between clay particles.

It was indicated that frictional resistance and thus shear resistance was directly related to the normal stress acting across the shear plane. R forces in the above equation were considered to be Coulombic pressures and A forces van der Waal's or secondary valence pressures. For most soils, (R-A) increases with decreasing interparticle spacing. In words, the equation states that in a highly plastic, saturated, dispersed clay, the effective stress is the net electrical stress transmitted between particles.

2.11 Cohesion, friction and dilatancy comprise Lambe's mechanistic components of shear resistance. Cohesion is the shear resistance which can be mobilized between two adjacent particles which "stick, cohere" to each other without the necessity of any externally derived normal pressure. Cohesion may be due to cementation of particles and/or various electrostatic bonds such as salt flocculation, edge-to-face flocculation, H-bonding or K-bonding. It was proposed that cohesion

was mobilized at very small strains, after which it was destroyed and no longer contributed to shear resistance. Dilatancy due to particle interference and friction formed Lambe's mobilized shear resistance which was a direct function of the normal force on the shear surface. Part of the friction component was considered to be electrical in nature. The dilantancy component disappeared at that strain where no further tendency toward volume increase was evident and then the friction component, including particle interference, became more important. Eventually, all shear resistance was attributed to the friction component.

Rosengvist's Concept of Shear Strength

2.12 Rosenqvist (1955) pictured clay minerals as being surrounded by adsorbed cations in a quasi-cystalline water layer. When clay minerals are brought together, the outermost layer of the mineral (the water film) yields plastically and little or no elastic stresses arise. Thus a subsequent release of the pressure does not bring the minerals totally apart, and they will, even in the unloaded state, stick together because of adhesional forces. This line of reasoning forms the basis for Rosenqvist's concept of cohesion. The adhesional forces in a clay-water system are considered to be van der Waal's in nature and depend upon the mutual distance between the minerals and the surface cations adsorbed upon them. It is postulated that van der Waal's forces are proportional to the polarizability of the adsorbed cations if the forces occur between cations adsorbed on one mineral and the lattice field of another mineral. If the forces occur between two

ions of the same kind, the forces are proportional to the second power of the polarizability. Thus the binding between clay minerals is caused by polarization of the adsorbed cations and the shear strength, which in Rosenqvist's view may be regarded as proportional to the cohesion, is dependent mainly upon the adsorbed cations. The frictional resistance in clays is made up of "macro-dilatency" and "micro-dilatancy", both proportional to the effective normal stress in t he shear plane. Rosenqvist felt at least one additional term was necessary to account for mass forces (electrical) between atoms which was a function of the normal stress. This was termed the "non-dilatant friction". The conclusion reached by Rosenqvist was that there was no fundamental difference between cohesion and friction. The shear strength was then simply presented as the effective sum of:

a) forces tending to bring the minerals closer together; i.e. effective normal pressure and van der Waal attraction, and

b) forces tending to separate the minerals; i.e., pore water pressure and electrostatic Coulomb repulsion.

2.13 Michaels (1959), in discussing Rosenqvist's concept of cohesion, expresses some extremely interesting views with regard to the role played by water in clay cohesion. It was postulated that the cohesive property of clays arose from interparticle adhesion and occurred in spite of, rather than because of, the presence of water. The water thus acted to reduce the forces of adhesion but water, being a high dielectric-constant media, allowed the particles to exhibit weak attractions to relatively large interparticle distances. Double-layer

forces which acted over large distances and were responsible for attraction between edges and faces of clay particles were thus responsible for the "card-house" structure normally associated with water sediments. The composition of the pore fluid and the type of exchangeable ions present determined the way in which interparticle attractions and repulsions guided or directed the geometric arrangement of primary particles. Thus, Michaels felt that the geometric arrangement of the particle was the primary strength determining variable. An explanation based on structure avoided the necessity of making assumptions as to the nature of adsorbed water, or to the role of electrolytes and exchangeable ions in influencing interparticle adhesion.

Schmid's Energy Concept

2.14 Schmid's basic picture of terraced clay minerals covered by chemisorbed ions or molecules is essentially similar to the common trend of thought. However, his interpretation of the nature of friction and cohesion exhibited by clay soils represents another approach to the explanation of these parameters. Friction was felt to be the result of the formation of hot and cold welded joints at the contact points of the clay minerals as sliding took place. With progressing shear strains, more and more welded contacts participated in resisting the shearing stresses until failure occured. The shear strength was thus determined by the type of mineral, the soils grains, the adsorbed water film, the geometry of the particles and the soil structure. It was reasoned that large amounts of residual contacts which were equivalent to an intrinsic pressure resulted when a clay

was unloaded. Thus the cohesion in the evaluation of shear tests appeared as a result of an intrinsic residual prestress whereas friction appeared as the result of active exterior forces causing a change in the composition (density) of the soil. It was concluded there was no difference, physically, between cohesion and friction. Schmid emphasized that the most important factor in determing strength was the composition and thus to obtain a material property one must keep the material itself constant. That is, strength tests on soils should be conducted under conditions that prevent void ratio or water content change. The standard test that approximately satisfies these requirements is the quick, undrained tests. Consolidation may be considered to result in a change in composition and thus results in changes in strength. According to Schmid, the energy adsorbed in the process of consolidation determined the number of contacts per unit volume and thus the yield strength of the material in shear. The best present measure of the number of contacts is the void ratio or water content for saturated soils. It was cautioned, however, that strength also depended on the duration of stress and the stress or strain rate.

Physico-Chemical Phenomena and Work Performed at the University of Alberta

2.15 The following paragraphs present a brief summary of the nature of clay-water systems and the characteristics of various cations in this system. Explanations of shear strength tests on homionic soils performed at the University of Alberta based on the concepts presented herein will also be reviewed.

2.16 The generally accepted basic unit in a soil-water system consists of an electrically unbalanced crystalline clay particle, a double layer made up of oriented water and adsorbed cations, and an outer liquid phase commonly known as pore water. The nature of the double layer depends upon the surface charge density of the clay mineral, its bonding energies, the kind of adsorbed ions and concentration of electrolytes in solution. The double layer consists of an inner immovable layer and an outer diffuse layer. The net negative charge of the particle is satisfied by adsorbed ions in the oriented water resulting in electrical neutrality and thus a potential drop across the double layer. The potential drop across the diffuse double layer is termed the 'zeta potential'. An increase in electrolyte concentration in the pore water tends to suppress the double layer and thereby reduce the 'zeta potential'.

2.17 Interparticle forces existing in a clay-water system are of primary importance and, according to Lambe (1958), are completely responsible for the shear strength of a clay. These forces may be divided first into those of attraction and of repulsion. The attractive forces present in the clay-water system are primarily secondary valence forces, also called van der Waals-London forces. These forces arise due to the presence of electrical moments in the individual molecules. The van der Waals forces between two atoms vary inversely as the seventh power of the distance between them while the attractive potential between two plates (or particles) varies inversely as the square of the distance between them (Lambe, 1953). The

hydrogen bond may also be considered a secondary valence bond but is stronger than the van der Waals forces. Other seats of attraction in the clay-electrolyte system are particle-cation, water-cation-water, and particle-water linkages. Repulsive forces exist between cations and between particles. The latter forces arise when the double layer of two particles interact. The nature of the double layer in a given soil-system is, as previously mentioned, primarily determined by the valence and concentration of the ions in the system. Repulsions increase at an exponential rate as the particles approach each other and a decrease in ion valence or ion concentration also causes an increase in repulsion for all but small intercolloidal distances. Lambe (1960) indicates that, for most soils, the net electrical force (R-A) increases with decreasing particle spacing. R is also considered to be relatively sensitive, but A insensitive, to system characteristics.

2.18 TABLE II.1 lists various characteristics exhibited by the four cations in clay-water systems selected for this program. The polarizability, which represents the extent to which an induced dipole is formed in the molecule, is a contributing factor to the van der Waals forces mentioned in the preceeding paragraph. The polarizability is of particular interest because it forms the basis for Rosenqvists' concept of shear strength (paragraph 2.12). Polarizability increases as the valence of the adsorbed cation increases and as the size of the cation decreases (Grim, 1953). High-atomic-number alkali ions have (because of reduced hydratability or increased

polarizability) a greater ability to reduce the zeta potential (Michaels, 1959) and thus increase strength.

2.19 Lambe (1958) indicates that "any change in the soil water system which expands the double layer tends to decrease soil strength (at a given void ratio), since interference by the double layers of two adjacent colloids increases interparticle repulsion. This concept leads to the prediction that any of the following changes would generally reduce the shear strength of a clay:

1. Reduction of electrolyte concentration.
2. Cation exchange from high to low valence (e.g., Ca^{++} to Na^{+}).
3. Adsorption of anions.
4. Exchange from a cation of small hydrated radius.
5. Increase of dielectric constant of pore fluid.
6. Increase of pH of pore fluid.
7. Decrease of temperature.
8. Increase in water content".

2.20 Paragraph 1.3 briefly outlined previous programs carried out at the University of Alberta illustrating physico-chemical effects. The strength tests carried out by Thomson (1963) on soil modifications whose predominant adsorbed cations were potassium, calcium, magnesium, and sodium yielded effective friction angles of 22°54', 22°42', 23°36', and 7°30' respectively. These values represent friction angles at an extrapolated zero salt content of the pore water; the extrapolation being necessary because salt content was not controlled. The above variations in strength were attributed to changes in thickness of the adsorbed water hull. A thick water hull was held responsible for the low strength of the sodium samples whereas the thin water hull associated with the other three types of modifications allowed greater particle interlocking and thus a greater strength was shown. An increase in salt concentration of the pore water was shown to

TABLE II.I

CHARACTERISTICS OF VARIOUS CATIONS

Cation	Calcium	Magnesium	Potassium	Sodium
Non Hydrated radius Å [1] Hydrated radius Å [1]	(1.17) (1.06) 9.6	(0.89) (0.78) 10.8	1.33 (5.32) (3.8)	0.98 (7.9) (5.6)
Zeta Potential of adsorbed cation [2]	52.7	54	56.5	57.7
Nature of oriented water generated by adsorbed cation [3]	well oriented to approx. 10 Å and transition to non-oriented abrupt	well oriented to slightly less than Ca	forms tight bond between particles, thus very thin oriented layer and well defined boundary	well oriented to approx. 7.5 Å and gradual transition to tens of molecular layers
Soil Structure Tendency[4] when cation is adsorbed	flocculated	flocculated	both dispersed and flocculated	dispersed
General Order of Replaceability [5]	1,2	2,1	3	4 (most easily replaced)
Polarizability, cm^3 [6]	--	--	$.987 \times 10^{-24}$	$.504 \times 10^{-24}$
Atomic No.	20	12	19	11

1. Grim (1953) p. 148
2. Bauer (1956)
3. Grim (1958) p. 20
   (1953) p. 174
4. Lambe (1958)
5. Grim (1953 p. 145)
6. Rosenqvist (1955)

increase the strength of a given modification. The reason for this increase was felt to be due to a suppression of the water hull. This effect was most pronounced with the sodium modifications and only slightly noticeable in other modifications. As a follow-up on Thomson's work, Locker (1963) performed strength tests on homionic calcium and sodium clay soils with controlled concentrations of pore water salts. An increase in salts in the pore water resulted in increased strength in the case of the sodium specimens and no significant change in strength in the calcium modifications. These effects were also explained in terms of water hull thicknesses. Strength variations were exhibited primarily in terms of friction angles with cohesion intercepts showing no consistent variation with salt concentration. Both workers showed that the specific gravity, liquid limit and consolidation times varied with the adsorbed cation and the salt content of the pore water. The results of Locker's work indicated that physico-chemical phenomena were more important in the region of high void ratios and low confining pressures.

CHAPTER III

SAMPLE PREPARATION AND THE TRIAXIAL TEST

Soils Used in Program

3.1 Material for tests on remoulded specimens was taken from a depth of approximately 6 feet from the surface on the south side of a road cut in Belgravia Ravine, Edmonton, Alberta (about 15 to 20 feet below the original ground surface). The material is a glacis-lacustrine sediment and is classified as a highly plastic clay. Classification tests in accordance with ASTM procedures yielded the results shown in TABLE III.1. The mineralogical composition of the clay fraction was determined by the Alberta Research Council on a soil similar to the one used for this program and is therefore only approximate.

3.2 A shelby tube sample taken from the Lendrum school playground, Edmonton, Alberta, from a depth of 14 to 15 feet was kindly provided by Bernard - Curtis - Hoggan Engineering and Testing Ltd. This material is geologically similar to the remoulded material tested for this program. The results of classification tests performed are shown in TABLE III.1.

Soil Modifications

3.3 The procedure followed in modification preparation was based

TABLE III.I

SUMMARY OF CLASSIFICATION TESTS

Test	Soil Used for Remoulded Specimens	Soil Used for Undisturbed Specimen
Specific Gravity	2.79	2.75
Atterberg Limits		
Liquid Limit	74.3	67.5
Plastic Limit	30.8	25.5
Plasticity Index	43.5	42.0
Grain Size Distribution [1]		
% Sand Sizes	4	22
% Silt Sizes	35	34
% Clay Sizes	61	44
Mineralogical [2] Composition		
Montmorillonite	30-40%	--
Illite	30-40%	--
Chlorite	20-30%	--

1. M.I.T. Grain Size Scale
2. As determined by Research Council of Alberta on clay size fraction - approximation.

on previous work by Thomson (1963) and Locker (1963). Air Dry soil was mechanically ground to pass a Number 40 seive. Two hundred gram batches were weighed out in one quart sealers. Seven hundred and fifty millilitres of 0.75N HCl was added slowly with constant stirring. Some reaction was evident and thus the mixture was allowed to stand for 5 to 10 minutes. The mixture was placed on a milkshake mixer for one minute and then allowed to stand for 24 hours. Care was taken to place the top loosely on the jar to allow the escape of any gases that may have been generated. All the soil had settled within the 24 hour period allowing the supernatant liquid to be siphoned off. The jar was refilled with approximately three hundred and thirty millilitres of 0.75N HCl and the above procedure repeated three more times. Following the last acid wash, the soil was washed twice with three hundred and twenty millilitres of distilled water to reduce the concentration of the remaining acid. After the final water wash, the jar was filled with a one normal solution of the acetate of the desired cation. The contents were mixed on the milkshake machine for one minute and then allowed to settle. The supernatant liquid was siphoned off and the jar refilled with acetate solution. The above procedure was repeated three more times resulting in the soil being washed with approximately 1250 millilitres of acetate solution. Twenty-four hours was found to be sufficient time in all cases for the soil to settle out. Following the last acetate wash, the contents of the jar consisting of 200 grams of soil and about 300 millilitres of acetate solution were transferred to 4 to 250 millilitres centrifuge bottles. The bottles were filled with

chemically pure ethyl alcohol, mixed on the milkshake machine for one minute and then placed in the centrifuge at 1300 rpm for 15 minutes. The supernatant liquid was poured off and the bottles refilled with alcohol. The contents were stirred with a glass stirring rod, as the settled soil was quite firmly packed, and then brought into suspension on the mixer. The soil in each bottle was treated with four washings of ethyl alcohol resulting in 40 to 50 grams of soil being washed with 600 to 650 millilitres of alcohol. The soil was removed from the bottles using fresh alcohol and dried at 55°C. Partial drying of the calcium soil revealed salt crystals on top of the soil and thus the soil was remixed, placed in the centrifuge bottles and rewashed twice more using about 150 millilitres of alcohol per bottle per wash. Three quart sealers of each type of modified soil were prepared yielding approximately 525 grams. The dried soil was crushed with a mortar and pestle to pass a Number 40 seive and thoroughly mixed. Sixteen grams of this were removed for cation exchange analyses; the remainder being split into three portions. Each portion thus obtained was sufficient to make one triaxial specimen.

Moulding Triaxial Specimens

3.4 Each portion obtained from the modification procedure was mixed into a slurry with distilled water to approximately the liquid limit, covered and stored in the moist room for at least one day. The slurry was remixed prior to placement in the one-dimensional consolidation molds. The consolidation apparatus consisted of two

concentric lucite tubes 2 3/4 and 1 3/4 inches in diameter, the smaller cylinder being 5 3/8 inches long. Four vertical, 1/2 inch wide, saturated, Whatman Number 54 filter paper strips were placed inside the inner tube and wedged in place at the bottom by means of a porous stone covered by a piece of filter paper. The slurry was placed into the inner mold with the aid of a spatula and tapped on the counter after each addition of soil to remove entrapped air. The mold was filled to approximately 1/2 inch from the top and then covered with a filter paper, porous stone and loading cap containing vertical drains. The annular volume between the cylinders was filled with water and the vertical load applied by means of weights on a lever arm. Consolidation was observed by means of dial readings plotted against the logarithm of time. Load increments of 0.36, 0.65 and finally 1.61 kg. per sq. cm. were applied, at least complete primary consolidation being allowed under each increment. The sodium modified specimens, because of their extremely long consolidation times, were loaded in increments of 0.36 kg. per sq. cm., each load application being left for about two days to prevent excessive squeezing of soil past the top loading cap under the next load. When the final pressure of 1.61 kg. per sq. cm. was reached, complete consolidation was allowed which required approximately one month. After consolidation under the last load had taken place, the apparatus was dismantled, the specimen extruded, wrapped in polyethylene plastic, waxed and stored in the moist room until required.

Mounting and Consolidating Triaxial Specimens

3.5 The procedure followed in trimming and mounting the triaxial specimens was similar to that described by Andreson et al (1957). A detailed description of the exact procedure followed is given by Locker (1963) and will only be summarized here. The specimen was trimmed to approximately 35.6 mm in diameter by 80 mm in length and then weighed and measured. Drainage aids consisted of five wool wicks inserted symmetrically into the specimen, five external filter paper strip drains (slotted filter paper) and filter paper on the top and bottom of the specimen. The prepared specimen was mounted on the triaxial base and enclosed in two rubber membranes (fixed in place with six O-rings) and two layers of silicone grease. The triaxial cell was filled with water topped with an oil seal, the confining pressure applied and the sample allowed to consolidate by expelling water into a 25 millilitre stopcock burette. Consolidation was performed under a single load increment and was followed by log and root time versus burette reading plots. When adequate secondary compression time was indicated, as determined on the log time curve, the burette was removed and the triaxial cell placed in the loading press.

The Triaxial Test

3.6 A detailed description of the procedures followed during back-pressuring, the pore pressure reaction test and the CFS test is given in APPENDIX A, consequently the treatment here will be brief. Back-pressuring was accomplished by, first, increasing the cell pressure by 2 kg. per sq. cm. and measuring the build up of pore

pressure with time for 15 to 20 minutes. This was essentially a pore pressure reaction test without back pressure. The pore pressure was then increased to match the back pressure and allowed to reach equilibrium conditions over a period of 12 to 15 hours. The standard pore pressure reaction test was then conducted which consisted of simply increasing the cell pressure by 1 kg. per sq. cm. and measuring the build up of pore pressure over a time interval of 5 minutes. The additional 1 kg. per sq. cm. was then released and the decrease in pore pressure measured over the same time interval. The CFS test was started in the same manner as an undrained test with pore pressure measurements in order to insure proper piston seating. As the test progressed at a constant rate of strain, the pore water pressure was varied to obtain a constant selected value of major principal stress ($\bar{\sigma}_1$). Data were obtained at this $\bar{\sigma}_1$, after which the pore pressure was changed to maintain a new constant value of $\bar{\sigma}_1$. The high and low values of $\bar{\sigma}_1$ selected for the majority of tests were 1.00 σ_c and 0.70 to 0.85 σ_c respectively. The procedure of "hopping" between the constant selected values of $\bar{\sigma}_1$ was continued past maximum deviator stress. Throughout the test pore pressure, deviator stress, volume change and time readings were taken. Plots of pore pressure, deviator stress, volume change, minor principal stress and effective stress ratio versus strain were kept as the test progressed. At the end of the test the entire assembly was dismantled, taking care to maintain equilibrium conditions within the specimen. Final wet weight, volume (by mercury immersion), slope of shear plane, if visible, and moisture content were noted. An example of the data for a complete triaxial test is given in APPENDIX C.

CHAPTER IV

DISCUSSION OF SAMPLE PREPARATION AND PRELIMINARY TEST PROCEDURES

Soil Modification

4.1 The object of preparing soils with a single cation adsorbed on the exchange complex and in the pore water is to magnify the effects of this single cation on the soil properties and to obviate any effects, known or unknown, other cations may have. The proportion of the cation exchange capacity of a soil that must be occupied by a single cation species in order to yield soil properties characteristic of this species only is uncertain. Work performed by Hamilton (1961) on an homionic calcium clay with various percentages of adsorbed sodium showed that a marked increase in plasticity characteristics occurred when 28% of the available exchange positions were filled with sodium. A further increase in percent adsorbed sodium, however, revealed a further gradual increase in plasticity characteristics to a maximum in the homionic sodium state. The soils prepared for this program were, on an average, 90% saturated with a given cation. The work of Hamilton illustrates that the characteristics of a given cation may be slightly modified by the presence of a small amount of other adsorbed cations. Following in this line of thought, it

might also be suspected that the presence of a small quantity of salts in the pore water would also affect the engineering characteristics of a given modification due to suppression of the double layer. Because the sodium cation in the adsorbed state is associated with a relatively large double layer, it would also be expected that this affect would be most pronounced with sodium modified soils. As the soil modifications produced all contained, according to exchange capacity analyses, a small quantity of salts, the preceeding thoughts should be kept in mind. Further reference to these ideas will be made in Chapter VII.

4.2 The first step in the preparation of a modification consisted of four washings with 0.75N hydrochloric acid. The purpose of these washings was to remove carbonates and sulphates from the soil. On one hand carbonates present in a soil tend to lower the cation exchange capacity as the large carbonate crystals occupy space in the soil mass that would otherwise be occupied by clay minerals. On the other hand, the presence of slowly soluble carbonates tends to increase the exchange capacity by acting as a source of calcium cations. Addition of water to an homionic soil containing carbonates will dissolve some carbonates resulting in a release of extraneous cations and a possible subsequent change the proportions of cations adsorbed.

4.3 Modified soils were prepared by washing with an acetate of the desired cation. This procedure was based on previous work (Thomson, 1963; Locker, 1963) which postulated exchange occurring in accordance with the Langmuir Adsorption Theory and the Law of Mass Action. As

indicated in Paragraph 4.1, the procedure used for this program did not yield completely homionic modifications as determined by flame photometer measurements. Exchange capacity analyses showed an average of 90% saturation with a specified cation on a given modification. The results of the exchange capacity analyses are shown in Table IV.1.

4.4 Excess salts in the pore water were removed by ethyl alcohol washings. The major advantage of using the acetate radical in preparing modifications is its subsequent easy removal because of its solubility in ethyl alcohol. The washings used, however, appear to be insufficient, as 6 to 7 me./100 gms a.d.s salts were found in some modifications.

Exchange Capacity Analyses

4.5 Quantitative determinations of the type of adsorbed cations and those in the pore water were made using procedures employed by the Alberta Research Council, Soil Survey Section. The only deviation from these procedures was that 5 grams of soil passing the Number 40 seive were used instead of 20 grams of a coarser size. Total cations present, including both those adsorbed and those in the pore water, is determined by summing flame photometer results for each ion found in a solution leached from the original soil by ammonium acetate. The cation exchange capacity of the soil itself is found by a nitrogen determination on the leachate containing ammonium ions which were present in the adsorbed state after the first leaching procedure. The difference between the flame photometer results and the nitrogen determination is a quantitative measure of the soluble salts in the pore water. Results

TABLE IV.I

RESULTS OF CATION EXCHANGE ANALYSES

Modification	Cation Exchange Capacity me/100 gms a.d.s.	Total Cations Present me/100 gms a.d.s.	Potassium	Calcium	Sodium	Magnesium	Salt Content me/100 gms a.d.s.
			me/100 gms. a.d.s. (percent adsorbed shown in brackets)				
Natural	29.3	95.0	1.2 (*)	74.9 (*)	0.9 (*)	18.0 (*)	65.7
Potassium	31.1	38.0	34.7 (89.4)	0.4 (1.3)	0.3 (0.9)	2.6 (8.4)	6.9
Calcium	34.0	40.0	0.6 (1.8)	37.1 (91.5)	0.3 (0.9)	2.0 (5.8)	6.0
Sodium	36.1	39.0	0.9 (2.5)	0.3 (0.8)	35.4 (90.0)	2.4 (6.7)	2.9
Magnesium	34.0	40.5	0.7 (2.0)	0.2 (0.6)	0.1 (0.3)	39.5 (97.1)	6.5

*Not determined

of the exchange capacity analyses performed for this program are shown in TABLE IV.1.

Moulding Triaxial Specimens

4.6 All specimens were placed in the one-dimensional consolidation apparatus at approximately their liquid limit. As the Atterberg limits were not performed on the modified soils because of the lack of prepared soil, the results of the limits performed by Hamilton (1961) were used as close approximations. Consolidation under a vertical pressure of 1.61 kg. per sq. cm. completed the moulding procedure. As initially identical specimens are a prerequisite to later effective comparisons, it was hoped that the moulding procedure would yield similar specimens. Initial void ratios, listed in TABLE V.1, agree remarkable well within a given soil series. One might then safely assume that initial soil structures were also very nearly identical and thus the moulding technique appears to be satisfactory for the purposes of this program.

Drainage Aids

4.7 Five internal wool wicks, external slotted filter paper and top and bottom filter papers were used to aid both drainage and pore pressure equalization within the specimen. These aids are necessary, particularly in the performance of the CFS - test, to speed up movement of water and equalization of pore pressure. The disturbance due to insertion of the wool wicks is an undesirable feature but subsequent consolidation would appear to rectify any detrimental

effects. The particular advantage gained by the use of internal wool wicks was illustrated by Schmertmann (1962), who found that the use of 5 wicks, as opposed to none, increased the rate of consolidation 6 times. The pattern in which the wicks were inserted, one in the centre and 4 placed symmetrically around it, resulted in the longest drainage path being about 0.9 cm.

The Load Cell

4.8 The triaxial equipment used is designed such that the loading table moves up at a constant rate against a proving ring. The rate of strain in the specimen varies with the manner in which the table movement is shared between the specimen and the proving ring. In the early stages of a test, a high proportion of the table movement is used to compress the proving ring whereas in the high-strain portion of a test, almost all movement is used in compressing the specimen. During the CFS test shifting from one $\bar{\sigma}_1$ curve to another is required and these shifts would be accompanied by compression or extension of the proving ring, thus the rate of strain is not constant. The difficulties encountered when performing the CFS test using a proving ring were illustrated by Thomson (1963). Compression and extension of the proving ring and the associated period of time required for transfer of strain-energy from the proving ring to the specimen resulted in poorly defined deviator stress vs strain curves and also "saw-tooth" cohesion and friction vs strain curves (Thomson, 1963, Page A 40 and A 41). In order to eliminate possible complications due to changes in the rate of strain and to improve the simplicity with which the CFS test is performed, Schmertmann and Osterberg employed a load cell which was much less

compressible than the proving ring but retained sufficient accuracy to determine the applied load. The load cell yielded a rate of strain which was very nearly constant.

4.9 The load cell used for the program of this thesis was an aluminum tube, 6 inches long, 3/8 inches O.D., and 0.030 inches wall thickness. The centre two inches was machined to a reduced wall thickness of 0.019 inches. Two SR-4, Type A-7 strain gauges were mounted longitudinally on this portion of the tube and were connected in series. Using a strain indicator and constant stress calibration apparatus, the load cell was calibrated against known dead loads. The calibration curve obtained based on both loading and unloading cycles, was linear. The sensitivity obtained over the calibrated range was 0.0784 kg. per microinch/inch strain which, in terms of specimen cross-sectional area, is approximately 0.008 kg/sq. cm. per microinch/ inch strain. This sensitivity is considerably lower than that obtained by Schmertmann and Osterberg (1960, page 657). The probable reason for this may reside in different types of aluminum since approximately the same size load cell was used in both cases. The sensitivity was, however, considered adequate and acceptable for the purposes of this program. The maximum compression of the load cell for this program was in the order of .03 mm, that is, approximately 575 microinches/inch, the calibration being ultimately carried out to 636 microinches/inch. In contrast, proving ring compressions normally range from 1 to 3 mm. The accuracy and errors associated with the load cell used will be discussed in the following chapter.

4.10 The load cell used for the first test in this program (Test Number 1 in TABLE V.I) had a sensitivity of 0.151 kg. per microinch/inch strain. It was decided that this was inadequate hence a second load cell was constructed and calibrated to 500 microinches/inch strain. It was felt that this strain covered the anticipated range and Test Number 2 was performed. During this test, strain exceeded 500 microinches/inch and the calibration of the load cell changed by .002 kg. per microinch/inch. The change could result in an error of approximately 0.2 kg/sg. cm. in the calculated deviator stress for the test. The load cell was re-calibrated to 636 microinches/inch which covered the range of strains for all subsequent tests. Periodic checks on the calibration of the load cell indicated no further changes throughout the program. Zero readings on the strain indicator before and after a given test checked satisfactorily; thus no problem arose because of this factor. It should be noted that once the strain indicator is connected to the system and a zero reading is taken, the indicator must not be removed from the system until the test is finished. Zero readings for tests depend on the tightness of connections and thus vary among different test set-ups. Differing tightness of connections affect the resistance of the circuit and hence the zero reading.

Backpressure and Pore Pressure Reaction

4.11 Immediately after consolidation had taken place a pore pressure reaction test was performed which consisted simply of increasing the cell pressure by 2 kg/sq. cm. and measuring the increase in pore pressure over a period of 10 minutes. The procedure is described in

detail in APPENDIX A. The results of such tests are given in TABLE IV.2. It is seen that, even though initial degrees of saturation are very high, the response of the pore pressure measuring system to the increased cell pressure is extremely poor, particularly with the more impermeable potassium and sodium modified soils. Just why this does occur is not readily evident as high degrees of sturation are normally associated with high reactions. It must be kept in mind, however, that the figures shown represent the response of the entire pore pressure system to the increase in cell pressure and not only that of the specimen. When the pressure increment is applied to the specimen by increasing the cell pressure, and equal and opposite pressure attempts to arise within the specimen and the measuring system. This increased pressure will tend to dissolve any entrapped air and also cause expansion of the pore pressure lines. If the reaction is still not equal to the pressure increment, additional specimen consolidation must be initiated under the pressure difference remaining. Expansion of lines was shown by a separate test to be very significant under the first 2 kg/sq. cm. pressure increment. This expansion was found to be in the order of 0.20 c.c and is probably the major contributing factor to the low reactions determined. The fact that lower values were obtained for the more impermeable specimens may be due to consolidation being much slower in these soils. In the case of the sodium modifications, the consolidation initiated during the short period required for the pore pressure reaction test is most likely negligible.

4.12 Following the preceding pore pressure reaction test, the pressure

TABLE IV.2

RESULTS OF PORE PRESSURE REACTION TESTS

Modification	Test Lateral Pressure Kg./sq.cm.	Initial Degree of Saturation %	Pore Pressure Reaction Without Backpressure in 10 min. ($\Delta\sigma_c = 2$)	Pore Pressure Reaction With Backpressure in 5 min. ($\Delta\sigma_c = 1$)
Nat. Rem.	2.10	97.8	*	0.95
Nat. Rem.	4.00	96.4	*	0.89
Nat. Rem.	4.00	97.5	1.13	0.81
Nat. Rem.	6.10	96.9	*	0.99
Nat. Undist.	4.00	99.0	1.39	0.87
Magnesium	2.00	98.8	*	0.96
Magnesium	4.00	99.2	1.30	0.99
Magnesium	6.00	98.9	1.00	0.95
Potassium	2.00	99.2	*	0.92
Potassium	4.00	98.8	0.60	0.81
Potassium	6.00	98.9	0.70	0.82
Calcium	2.00	98.9	1.54	0.98
Calcium	4.00	99.5	*	0.85
Calcium	6.00	99.2	0.75	0.89
Sodium	2.00	100.1	0.30	0.38
Sodium	4.00	100.7	0.19	0.24
Sodium	6.00	100.9	0.28	0.14

*Not determined

within the specimen was set at 2 kg/sq. cm. In this manner backpressuring was accomplished. This procedure is also outlined in detail in APPENDIX A. In the last five tests of this program, volume change measurements were made during this procedure and essentially negligible volume changes were found to occur within the specimen during back-pressuring if expansion of the volume change indicator and pore pressure lines was taken into account. The specimen was left under back pressure for 12 to 15 hours and a second pore pressure reaction test was performed. This test consisted of increasing the cell pressure by 1 kg/sq. cm. and measuring the response of the pore pressume system over a 5 minute period. Rsults of these tests are listed in TABLE IV.2. All reaction are in the range 80 to 99% with the exception of the sodium modifications. One would expect the reactions to be high be-cause of the increased saturation due to the back pressure (Lowe and Johnson, 1960). It was found from the separate test mentioned earlier that pressure increments subsequent to the first 2 kg/sq. cm. increment resulted in a volume expansion of the system in the order of 0.07 c.c. per 1 kg/sq. cm. This factor may contribute to values being less than 100%. Any consolidation which would be initiated would tend to counter-act the effect of expansion of the system. As noted earlier, the consolidation effect may be entirely absent in the sodium modifications during the short time interval used for these pore pressure reaction tests. The occurrence of low pore pressure reactions with sodium modified soils is not unique to this program but was also encountered in the programs of Thomson (1963) and Locker (1963).

4.13 Most final degrees of saturation were found to be larger that the initial values but this increase cannot be attributed solely to back pressuring as movement of water into and out of the specimen is allowed throughout the CFS test. Thus the effect of back pressure on the degree of saturation of a specimen cannot be quantitatively evaluated.

CHAPTER V

PRESENTATION OF DATA AND DISCUSSION

OF THE CFS TRIAXIAL TEST

General

5.1 The problem forming the basis of this thesis was to investigate the variation of the strength parameters, cohesion and angle of internal friction, with axial strain in modified clay specimens using the CFS test. The terms "cohesion" and "angle of internal friction" or "friction" are retained in the presentation and discussion of the results of this program to be consistent with the original definitions put forth by Schmertmann and Osterberg (Paragraph 2.6) although preference has been expressed for the phrases "angle of shearing resistance" or "shearing resistance" to replace the latter terms. Curves of imposed axial stress, herein referred to as deviator stress, versus strain for all tests are in APPENDIX B. Plots obtained from consideration of these curves are presented in this and the following chapter. Complete sets of data which illustrate in detail CFS test procedures and calculations are presented in APPENDIX C.

Calculations

5.2 Initial void ratio calculations were based on determinations of the initial moisture content from specimen trimmings, the volume

from measurements of the specimen and average specific gravity of soil solids from at least two determinations. These values are listed in TABLES V.1 and V.2. The decrease in volume of the specimen resulting from triaxial consolidation was measured by the quantity of water expelled into a 25 ml. burette. This enabled the calculation of volume, moisture content and void ratio after consolidation had occurred. Plots of consolidation pressure versus moisture content after consolidation for each modification were found to be generally poor due to scatter of points from the accepted straight line relationships. Determinations of the volume change during any given CFS test based on the above procedure of calculating moisture contents after consolidation appeared to be very high when compared to actual measurements made with the volume change indicator in the pore pressure line (APPENDIX A, Method 2).

5.3 Thomson's work (1963, page A21) with undrained strength tests also revealed, in some instances, a very marked discrepancy between the initial wet weights and the sum of final wet weights and burette volume changes. Hence there appeared to be adequate justification for suspecting burette volume changes and also for correction of these measurements if a suitable method could be devised. This problem did not require special attention in the work of Thomson (1963) and Locker (1963) as the necessity of calculating moisture contents after consolidation from burette volume changes did not arise. As the tests used were undrained, final moisture contents equalled those after consolidation.

5.4 It was decided to perform a simple triaxial consolidation test on the remoulded Edmonton clay to investigate this problem. The

TABLE V.1

SUMMARY OF TEST DATA

TEST NO.	SOIL TYPE	σ_c KG/SQCM	Gs	CONSOL. TIME MINUTES t_{90}	CONSOL. TIME MINUTES t_{100}	SECOND. CONSOL. TIME MINUTES	MOISTURE CONTENT, % INITIAL	MOISTURE CONTENT, % AFTER CONSOL.	MOISTURE CONTENT, % FINAL	VOID RATIO INITIAL	VOID RATIO AFTER CONSOL.	VOID RATIO FINAL	FINAL DEGREE OF SATUR.	RATE OF STRAIN MINUTES FOR 1 %	STRAIN %- END OF TEST	CONST. $\bar{\sigma}_1$ KG/SQCM	AT MAX. DEVIATOR $\sigma_1 - \sigma_3$ KG/SQ CM	AT MAX. DEVIATOR u KG/SQ CM	AT MAX. DEVIATOR STRAIN %	PEAK c_ϵ KG/SQCM	MAX. ϕ_ϵ DEG.	SHEAR PLANE ANGLE DEG.
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
3	NAT. REM.	2.10	2.79	449	510	2451	47.98	38.95	40.65 *	1.33	1.12	1.133 *	100.1	155	16.0	2.36 1.80	1.36 1.145	1.07 1.68	6.0 6.5	0.450	12°33'	*
1	NAT. REM.	4.00	2.79	92	210	1400	45.91	34.44	* 34.96	1.33	1.01	0.991	97.5	96	14.6	4.30 3.60	2.71 2.47	2.23 2.84	9.5 9.4	0.795	14°00'	59
13	NAT. REM.	4.00	2.79	72	205	2635	47.23	34.47	36.02 36.61	1.35	1.00	1.026 1.042	98.0	155	11.1	4.00 3.25	2.33 2.085	2.33 2.81	7.0 6.7	0.710	12°24'	60
2	NAT. REM.	6.10	2.79	98	220	1530	46.25	31.69	31.63 *	1.37	0.93	0.905 *	97.5	155	16.2	6.30 5.30	3.47 3.19	3.30 4.00	8.8 7.3	1.360	14°30'	57
14	NAT. UND.	4.00	2.76	37	89	2871	37.90	32.41	34.89 35.42	1.06	0.91	0.982 0.996	98.1	155	6.3	4.00 3.26	2.51 2.24	2.51 2.98	4.6 4.2	0.790	14°25'	60
4	MG	2.00	2.69	123	355	1445	52.90	44.80	45.00 45.75	1.44	1.23	1.213 1.237	99.5	155	12.8	2.36 1.77	1.315 1.16	0.95 1.38	7.0 6.5	0.496	11°30'	58
7	MG	4.00	2.69	90	212	3088	52.87	38.46	39.72 40.34	1.43	1.05	1.067 1.087	100.1	155	11.6	4.00 3.25	2.36 2.07	2.36 2.80	9.6 8.5	0.782	14°29'	64
10	MG	6.00	2.69	205	500	3070	52.72	34.77	36.75 37.18	1.43	0.95	0.984 0.996	100.5	155	11.6	6.04 5.12	3.28 2.975	3.25 3.85	8.6 9.3	1.066	11°48'	61
5	K	2.00	2.72	428	630	2550	51.74	43.61	46.37 46.80	1.42	1.20	1.255 1.275	99.8	155	11.9	2.04 1.55	1.40 1.14	1.36 1.60	8.5 7.8	0.421	20°52'	64
8	K	4.00	2.72	740	1440	3180	49.86	37.57	39.60 39.96	1.37	1.04	1.076 1.088	100.1	155	10.8	4.04 3.38	2.52 2.28	2.48 2.91	7.0 6.0	0.813	17°04'	62
11	K	6.00	2.72	322	660	4040	51.90	37.04	38.47 38.85	1.43	1.02	1.023 1.034	102.2	155	10.0	6.04 5.18	3.925 3.575	3.88 4.38	7.5 6.2	1.280	17°45'	66
6	CA	2.00	2.72	65	285	3135	52.78	44.58	46.10 46.92	1.45	1.23	1.266 1.288	99.1	155	10.4	2.04 1.54	1.26 1.10	1.22 1.61	7.4 5.5	0.406	13°20'	61
9	CA	4.00	2.72	80	235	3125	52.56	39.16	40.41 41.08	1.44	1.07	1.100 1.122	99.6	155	10.4	4.00 3.24	2.275 2.02	2.275 2.75	7.6 7.4	0.701	11°40'	*
12	CA	6.00	2.72	82	173	3017	53.35	36.84	37.12 37.57	1.46	1.01	1.017 1.029	99.3	155	10.1	6.05 5.12	3.405 3.11	3.35 3.99	8.9 9.0	1.107	10°52'	*
15	NA	2.00	2.75	51,980	54,000	12,000	75.56	61.58	69.67 69.96	2.08	1.69	1.919 1.927	99.8	1393	5.2	1.97 1.46	0.75 0.73	0.79 1.29	2.9 2.9	0.340	2°00	68
16	NA	4.00	2.75	63,500	70,000	11,000	73.76	53.69	55.75 55.81	2.01	1.46	1.606 1.608	95.5	1393	5.4	4.00 3.62	1.54 1.51	1.55 1.88	3.4 3.4	0.638	2°21	61
17	NA	6.00	2.75	80,800	72,000	8,800	71.83	48.29	48.83 48.90	1.96	1.31	1.349 1.351	99.6	1393	5.4	5.98 5.61	2.07 2.04	2.08 2.44	4.9 4.7	0.810	2°40	*

8, 11, 15, 16, 17, 18 – UPPER AND LOWER NUMBERS IN A ROW CORRESPOND TO HIGH AND LOW CONSTANT $\bar{\sigma}_1$ CURVES RESPECTIVELY

* – NOT DETERMINED

TABLE V.2

SUMMARY OF VOLUME CHANGE DATA

SOIL TYPE	σ_c KG/SQ CM	VOLUME CHANGE-BURETTE C.C.	THREE MINUTE CORRECT. C.C.	CORRECT. VOLUME CHANGE C.C.	ORIG. VOLUME SPEC. C.C.	FINAL VOLUME SPEC. C.C.	VOLUME CHANGE DURING TEST C.C.	INITIAL WET WEIGHT GMS	FINAL WET WEIGHT GMS	VOLUME CHANGE DURING TEST C.C.	INITIAL WEIGHT WATER GMS	FINAL WEIGHT WATER GMS	VOLUME CHANGE DURING TEST C.C.	AVERAGE VOLUME CHANGE CFS HOP C.C.	
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
NAT. REM.	2.10	9.37	0.50	8.87	83.39	75.11	0.59	145.35	137.53	1.05	47.13	39.93	1.67	*	U
NAT. REM.	4.00	14.80	3.39	11.41	83.08	71.03	-0.64	145.19	133.60	-0.18	45.68	34.79	0.52	*	L
NAT. REM.	4.00	15.36	2.80	12.56	82.94	72.05	1.67	144.92	134.08	1.72	46.49	36.04	2.11	0.59	L
NAT. REM.	6.10	16.02	1.63	14.39	82.51	67.43	-0.69	144.42	129.76	-0.27	45.68	31.23	-0.06	*	U
MG	2.00	8.23	0.78	7.45	83.43	76.37	0.39	140.43	133.55	0.57	48.59	41.71	0.57	0.82	L
MG	4.00	15.64	2.40	13.24	83.05	70.55	0.74	140.36	127.73	0.61	48.55	36.47	1.16	0.58	U
MG	6.00	17.98	1.50	16.48	83.10	67.71	1.09	140.24	125.17	1.41	48.41	33.75	1.82	0.40	U
K	2.00	8.20	0.68	7.52	82.22	77.35	2.65	140.33	134.93	2.12	47.85	43.28	2.95	0.67	L
K	4.00	12.30	0.58	11.72	83.19	72.77	1.30	144.02	132.38	0.08	47.54	37.75	1.93	0.40	U
K	6.00	15.34	1.44	13.90	83.49	69.94	0.35	142.08	128.82	0.64	48.55	36.34	1.69	0.36	L
CA	2.00	9.00	1.46	7.54	82.86	77.32	2.00	140.42	134.76	1.88	48.51	43.12	2.15	0.75	L
CA	4.00	14.40	2.00	12.40	82.90	72.18	1.68	141.18	129.90	1.12	48.64	38.02	1.78	0.58	L
CA	6.00	17.55	2.40	15.15	83.03	68.00	0.12	140.67	125.61	0.09	48.94	34.05	0.26	0.42	U
NA	2.00	10.80	0.42	10.38	83.04	79.00	6.34	130.32	125.63	5.69	56.09	51.93	6.22	0.21	L
NA	4.00	15.51	0.30	15.21	83.09	71.90	4.02	131.75	117.65	1.11	55.92	42.32	1.61	0.11	L
NA	6.00	18.50	0.37	18.13	82.82	65.85	1.16	132.37	114.55	0.31	55.33	37.67	0.47	0.05	L
NAT. UND.	4.00	7.20	1.05	6.15	83.47	80.43	3.11	154.51	151.30	3.15	42.46	39.09	2.78	0.59	U

1– TRIAXIAL CONFINING PRESSURE

7, 10, 13– NEGATIVE VALUES REPRESENT VOLUME DECREASE

*– NOT DETERMINED

FIGURES IN COLUMNS: 6 + 4 – 5 = 7; 9 + 4 – 8 = 10; 12 + 4 – 11 = 13

15– U– UPPER $\bar{\sigma}_1$ CURVE

L– LOWER $\bar{\sigma}_1$ CURVE

difference between initial and final weights of water, volumes, wet weights and burette readings could then be compared to determine the reliability of the burette method of calculating volume change. Each of the preceding methods of calculating volume change involve a separate, independent factor and thus may be considered unrelated for comparison purposes. The results of such a test did show the burette volume change measurements to be in error by a significant quantity. The difference between initial and final weights, volumes and weights of water agreed within 0.50 c.c. in 14.50 c.c. whereas burette volume change was 2.00 c.c. high.

5.5 Some of the excess water expelled during consolidation may be that used to saturate the drainage aids described in Paragraph 4.7. Air was also expelled and is included in the measured volume change. This air may have been trapped between the membrane and the specimen or during insertion of the internal drains when the sample was set up and thus would also contribute to erroneous measurements.

5.6 The only means available on which to base a correction was the comparison of the triaxial consolidation curve obtained on the one remoulded Edmonton clay specimen and the individual consolidation test results on this clay from the CFS tests. A comparison of these indicated that the volume correction necessary to make the individual test results agree with the assumed correct triaxial consolidation curve could be obtained by subtracting the volume of water expelled after 3 minutes from the total volume expelled. The

value of 3 minutes was the average of 4 determinations; 0.50, 6.5, 3 and 2.5 min. from tests 1 to 4 respectively. The 3-minute correction obtained in this manner was then applied to all burette volume change measurements. The corrections obtained are shown in TABLE V.2 and values of moisture content and void ratio after consolidation based on corrected volume changes are listed in TABLE V.1.

5.7 Any future work requiring calculation of moisture contents after consolidation based on burette volume change measurements should attempt to obtain corrections to these measurements. Either or both of the following means may be sufficient to determine this correction. An initial consolidation under a very small load, for example, .05 kg/sq. cm., may expell any excess air or water present in the system. During consolidation under pressures such as those used for this program, a record of the quantity of air expelled should be made and applied as a correction.

5.8 Following the triaxial test, determinations of the specimen volume by mercury immersion, wet weight and moisture content were made. The change in volume during the CFS test could then be computed. The results of calculations by three methods are shown in TABLE V.2. All methods utilize the corrected burette volume change. Agreement between the three methods is as good as may be expected. Generally two methods, not necessarily the same two for every test, compare quite closely. The third method, in these cases, was probably the result of technical difficulties. For example, the final wet weight of the

specimen may be low due to loss of soil on the outer drainage aids when these are removed. Volume change measurements made during tests with the volume change apparatus in the pore pressure line (APPENDIX A, Method 2) indicated that the maximum volume increase occurred at approximately the same strain as maximum deviator stress. Following this the specimen volume tended to decrease very slowly (see example, APPENDIX C, Remoulded Edmonton Clay). Thus, the volume change recorded after a CFS test is dependent on strain, and this must be kept in mind when comparing values for different tests listed in TABLE V.2. The volume change tabulated also corresponds to either the high or low $\bar{\sigma}_1$ curve chosen, this factor being noted in the final column in TABLE V.2. The average volume change during the CFS test under the end-of-test strains employed for this program ranged from -0.8% to +3.8%, the net volume decrease generally corresponding to tests taken to high percent strains.

Measurement and Plotting Errors

5.9 All computations for this program were performed on a 12 place desk calculator and thus no errors should arise because of inaccurate calculations. Plotting errors are always present but are considered to be comparatively minor. The experimental measurements directly involved in the computations of the friction and cohesion components at any strain are the triaxial cell pressure, σ_c , the deviator stress, ($\bar{\sigma}_1 - \bar{\sigma}_3$), and the pore pressure, u. The triaxial cell pressure was maintained by a hydraulic weight-loaded-piston pressure cell. The deviator stress was measured by an external, calibrated load cell and strain indicator (see Paragraph 4.9) and pistion friction was minimized

by rotating piston bushings. The pore pressure was measured with mercury manometers having a maximum working range of 1.75 kg/sq. cm. For pressures greater than this, the range was extended by additional backpressure from an hydraulic pressure cell. An important source of potential error is that the controlled and measured pore pressure may not be the one actually effective throughout the specimen. The drainage aids (Paragraph 4.7) and test technique (APPENDIX A) are designed to minimize this error.

5.10 TABLE V.3 presents the author's estimates of the ordinary errors present in the significant variables when the stresses are at the levels used in the experiments reported herein. TABLE V.4 indicates the range in typical computed values of cohesion and friction as a result of the most unfavorable combination of errors in TABLE V.3.

Rate of Strain

5.11 It has been generally agreed (Bishop and Henkel, 1962; Schmid, 1962; Hvorslev, 1960; Taylor, 1948) that the strength of a normally consolidated clay is dependent to some extent on the rate of strain. Bishop and Henkel (1962, page 192) state:

"The rate of testing has a marked influence, in clay soils in particular, on the pore pressures observed during the undrained stage of the triaxial test. This arises from three causes:

a) time lag in the pore pressure device,
b) progressive equalization of nonuniformity in pore pressure resulting from end restraint or from a natural tendency to zone failure, and
c) a modification in the behavior of the soil structure as the rate of shear is reduced.

In choosing a rate of testing it is necessary to be able to determine

TABLE V.3

PRECISION OF MEASUREMENTS

Stress Measured	Ordinary Range of Error Deviation from true, kg/sq.cm	
	Below	Above
σ_3	0.00	+0.02
u	-0.01	+0.02
σ_d	-0.02	+0.03

TABLE V.4

RANGE OF COHESION AND FRICTION

DUE TO ERRORS SHOWN IN TABLE V.3

	Computed Minimum	Assumed True	Computed Maximum
Cohesion, C_ϵ kg/sq.cm			
at Low Strain*	0.624	0.717	0.817
at High Strain*	0.538	0.639	0.743
Angle of Internal Friction, ϕ_ϵ^o			
at Low Strain	$03^o46'$	$05^o55'$	$08^o05'$
at High Strain	$14^o59'$	$17^o43'$	$20^o33'$
Tan ϕ_ϵ			
at Low Strain	.0658	.1036	.1420
at High Strain	.2676	.3195	.3749

* Low Strain - less than 3%
* High Strain - at or above strain of max. ($\sigma_1 - \sigma_3$)

the relative influence of these three factors on the observed values of pore pressure, so that the results obtained will lead to a true measure of the physical properties of the soil."

5.12 In addition to the preceding factors, a rate of strain for the CFS test must allow for volume changes that take place. Schmertmann (1962) suggested that the CFS test be performed with a compression rate not greater than one percent axial strain per theoretical 100% consolidation time interval (t_{100}). As the first consolidation test for this program yielded a t_{100} of 210 minutes, a CFS test performed at the suggested rate would have taken at least 48 hours to ensure exceeding the strain at which maximum deviator stress occurs. This means that the test would have be be left unattended for two nights. In order to realize a substantial saving in time, it was decided to perform this first test at approximately 1% axial strain per 100 minutes (approximately ½ t_{100}) with the hope of still obtaining adequate pore pressure equalization and clear definition of the curves. The data from this test are shown in TABLES V.1 and V.2 and the deviator stress versus strain curves are given in APPENDIX B. It was found that this rate of strain was too fast resulting in poor definition of the deviator stress curves. The fact that this was the writer's first test also contributed to the erratic results. Technique is of vital importance in the performance of the CFS test and the mastering of this technique requires the conduct of pilot model tests.

5.13 All subsequent tests for this program, with the exception of the

tests on sodium soils, were conducted at a rate of strain of 1% per 155 minutes. Times to theoretical 100% consolidation varied from 89 to 1440 minutes, the majority being between 170 and 360 minutes. The tests appeared to be successful but fewer points for definition of curves were obtained for the more impermeable soils. CFS tests on the sodium modifications were conducted at 1% axial strain per 1393 minutes. Times to theoretical 100% consolidation for these modifications ranged from 54,000 to 72,000 minutes. Deviator stress versus strain plots from the sodium specimens tested are presented in Figure B.5 and a complete set of data for a test on the sodium modification is given in APPENDIX C. Difficulties were had with two of the three tests performed but it is felt that these arose mainly from problems concerning technique. The rate of strain was probably a contributing factor but cannot be considered wholly responsible.

5.14 The drainage aids employed for all specimens undoubtedly contributed largely to the success of the CFS tests reported herein. Possibly even more internal wool wicks utilizing different insertion patterns could have been used in the sodium specimens without any significant detrimental effects on the compressive strength determinations. This point should be kept in mind for any further CFS tests on soils similar to the sodium modifications used in this program.

5.15 The effect of rate of strain on CFS test results was examined by Schmertmann (Closure, 1962). Tests were performed on machine-extruded specimens of saturated kaolinite with strain rate as the only variable. The ratio between the fastest and slowest test was over

5000 to 1 (4.34%/hr. to .00078%/hr.). It was found that cohesion was similar in magnitude for all tests in spite of the greatly increased time for pore pressure distribution. The shear resistance mobilized at a given strain was, however, found to increase with decreasing strain rate. Schmertmann's CFS tests on this clay (1960) were performed at approximately 0.75%/hr. At a comparable strain, the lowest and highest tangents of the angle of internal friction were approximately 60% (4.34%/hr.) and 140% (.00078%/hr.) of that obtained from the test performed at a rate of strain of 0.75%/hr. From this particular test series at the strain compared this is approximately $11^{\circ} \pm 4^{\circ}$ which compares with an ordinary experimental error of $\pm 2^{\circ}$. Hence it would appear that strain rate is not critical. The strain rate must still, however, be compatible with the permeability of the soil tested in order to perform successful CFS tests.

Failure Criterion

5.16 The failure criterion generally adopted in triaxial tests are maximum deviator stress or maximum principal effective stress ratio. The mechanics of the CFS test makes these two points coincident. The CFS test, however, yields the mobilized components of the resistance of a soil to shear strain and indicates two separate failures. At low strains a cohesion failure occurs followed by a cohesion plus friction failure at often much greater strains. The latter corresponds closely to the point of maximum deviator stress for the curve obtained when $\bar{\sigma}_1 \sim \sigma_c$. For the purposes of this program maximum deviator stress was chosen to represent the failure condition and all compressive strength

and Mohr failure plots have been defined on this basis.

The CFS Test

5.17 All factors concerning strain are based on the original length of the sample as correction curves established by the Norwegian Geotechnical Institute (NGI Number 45, 1957) are based on original length. The correction is obtained by entering a plot of correction versus burette volume change from consolidation. The correction is used in calculations of corrected cross-sectional area and deviator stress. The burette volume change used to obtain the correction was the original unaltered value and thus a small, relatively constant error is associated with all deviator stress calculations. Any attempt to rectify this situation would involve recalculation of all test data. It is felt that the error is small enough, however, to be ignored.

5.18 One might ask why major principal effective stresses were chosen to remain constant during a CFS test rather than minor principal effective stresses or normal effective stresses on the failure plane. Schmertmann and Osterberg's reasoning behind this choice was that it would involve the least void ratio change and also appeared more practical than the alternatives suggested.

5.19 During performance of the CFS test hops must be made between two adjacent constant major principal effective stress curves. A compromise must be reached between minimizing structural changes by minimizing changes in effective stress, and increasing accuracy by

increasing the change in effective stress and therefore the effects of this change that are to be measured. Schmertmann and Osterberg (1960) decided that a $\bar{\sigma}_1$ range of about 75 to 100% σ_c produced strength changes that could be interpreted with sufficient accuracy and yet involved only small void ratio changes (usually less than 1%).

5.20 The majority of tests (4 to 17 inclusive) for this work were conducted within the range of about 75 to 105% σ_c. The first 4 tests were performed using a high $\bar{\sigma}_1$ of 105 to 118% σ_c. The first point on the high $\bar{\sigma}_1$ curve, at approximately 0.2% strain, was obtained under undrained test conditions. For suceeding points the pore pressure was controlled and measured as outlined in APPENDIX A. After the first four tests were completed the question arose as to whether performance of the CFS test with the high $\bar{\sigma}_1 \sim \sigma_c$ would yield notably different results than when $\bar{\sigma}_1 > \sigma_c$. Analysis of the data did not reveal any significant discrepancies in results. The revised procedure was adopted for the remainder of this program because tests appeared to be easier to start if pore pressure was immediately altered and controlled.

5.21 The results of drained and undrained tests appear to be applicable to certain specific field conditions, for example, long and short-term stability of slopes. As the CFS test is neither drained nor undrained but is accompanied by a strain-dependent volume change (Paragraph 5.4) one might query its physical interpretation. The performance of curve-hopping between two selected major principal effective stresses is analagous to repeated rebound and recompression consolidation cycles. The reason for operating along the rebound-compression branches is to

minimize any gross change of structure (as measured by a change in void ratio). These branches are kept sufficiently near the void ratio and stress condition of the soil after normal consolidation to consider the structure that of a normally consolidated clay. It is not difficult to visualize increased pore pressures occurring in the field resulting in a decrease in effective stress and a subsequent small rebound and thus the CFS test procedure does appear to be physically applicable.

5.22 The significance of a laboratory test should not, however, be judged only by the apparent macroscopic similarity between it and a field condition or on its ability to supply meaningful, fundamental soil strength parameters which may be applied to problems involving a given soil under various conditions. It should also be judged on its merits as a research technique in an effort to present a more concise and comprehensive picture of the source and nature of shearing strength of cohesive soils.

5.23 During all the CFS tests performed for this program, plots of deviator stress, minor principal stress, effective stress ratio, pore pressure and volume change versus strain were kept. All deviator stress versus strain curves obtained are presented in APPENDIX B. The curves for the tests in which the major principal effective stress was approximately equal to the consolidation pressure of 4 kg/sq. cm. are shown in FIGURE V.1. The curves given are typical of those obtained on all soil modifications tested at the other confining pressures. Sodium modifications exhibited the lowest strengths and failure strains

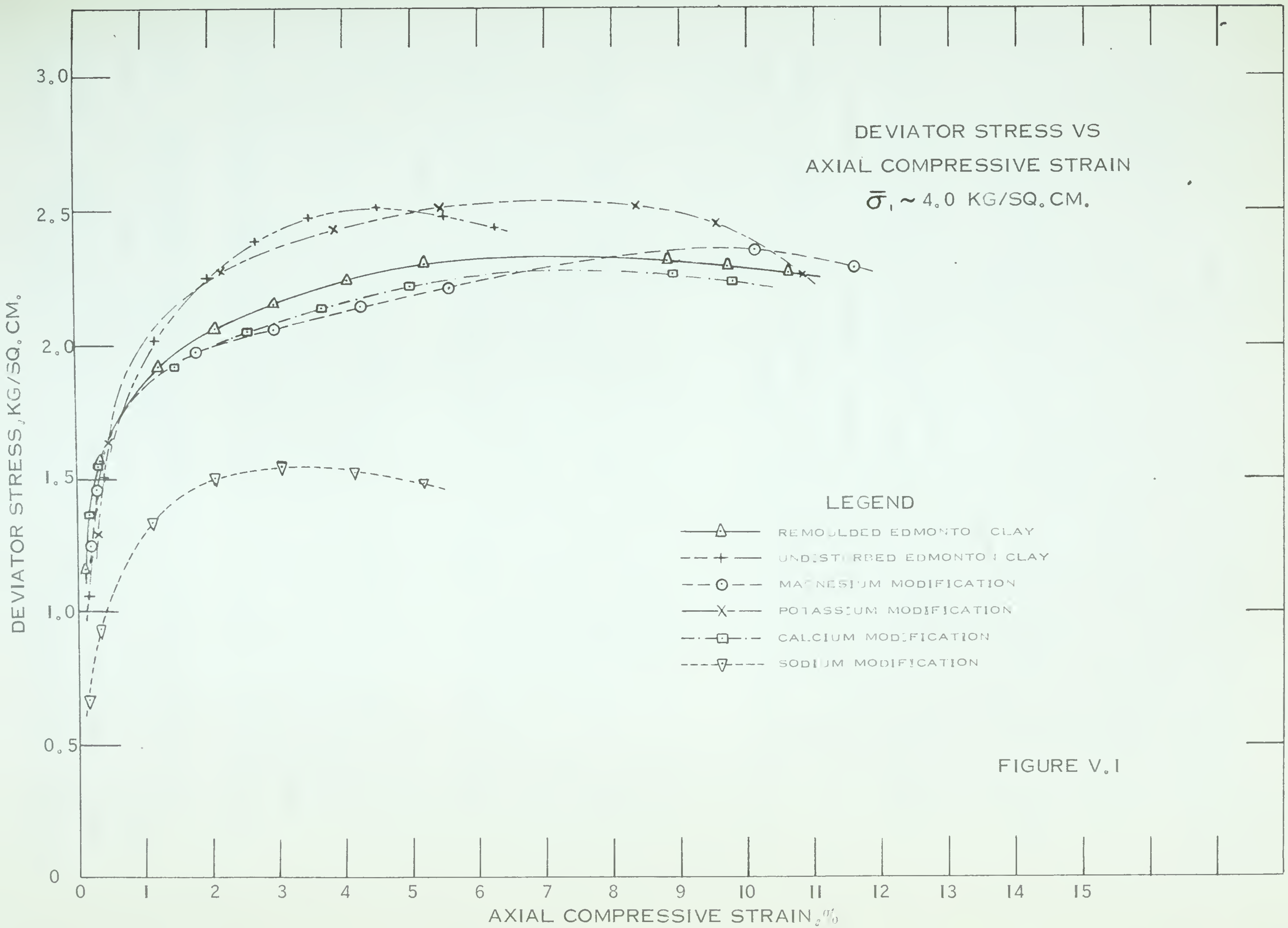

DEVIATOR STRESS VS
AXIAL COMPRESSIVE STRAIN
$\bar{\sigma}_1 \sim 4.0$ KG/SQ. CM.
LEGEND
REMOULDED EDMONTON CLAY
UNDISTURBED EDMONTON CLAY
MAGNESIUM MODIFICATION
POTASSIUM MODIFICATION
CALCIUM MODIFICATION
SODIUM MODIFICATION
DEVIATOR STRESS, KG/SQ. CM.
AXIAL COMPRESSIVE STRAIN, %
0
0.5
1.0
1.5
2.0
2.5
3.0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15

FIGURE V.1

at a given $\bar{\sigma}_1$ and consolidation pressure. The test on these modifications lasted approximately 6 days with one point on each constant $\bar{\sigma}_1$ curve being obtained each day after the first day of test. The undisturbed clay tested also failed at a low percentage strain, the test requiring only one day to complete under the rate of strain employed. The constant $\bar{\sigma}_1$ curves shown in Figure V.1 for magnesium and calcium modifications and for the natural soil are reasonably well grouped. The potassium modification yielded the highest compressive strength of all remoulded soils tested at this and other comparable constant major principal effective stresses. For the case illustrated in Figure V.1, the undisturbed clay shows a compressive strength similar to that of the potassium modification.

5.24 The stress-strain curve after maximum deviator stress for the potassium modification falls more rapidly than the other curves. From this observation, the potassium soil might be considered more brittle than the other soils tested. Strength tests performed by Rosenqvist (1955) on artificial potassium clays also illustrate the very brittle nature of these soils.

5.25 As $\bar{\sigma}_1 \sim \sigma_c$, pore pressure must be approximately equal to the applied deviator stress. Thus the curves presented in Figure V.1 can be thought of as pore pressure versus strain plots. The corresponding minor principal effective stress curves are thus found to be concave up.

5.26 Hops between two adjacent constant $\bar{\sigma}_1$ curves performed by controlled pore pressure changes result in a change in volume of the specimen and thus a change in void ratio and moisture content. Schmertmann (1960) reasons that two identical clay specimens at an identical strain condition,

but with different values of $\bar{\sigma}_1$ cannot have exactly the same structure because a void ratio change must occur for the following two reasons:

a) "As the specimen strains the particle alignment (fabric) is continually changing to accomodate strain. Since the clay is in equilibrium with a given intergranular stress only at a particular fabric and void ratio, any change in fabric will require a void ratio change to maintain the intergranular stress. Then, it follows that any intergranular stress change at a certain fabric (strain) requires a void ratio change.

b) During the compression the pore pressure is increased to maintain $\bar{\sigma}_1$ constant. Since σ_3 is held approximately constant then $\bar{\sigma}_3$ must be continually decreasing. This represents a reduction in the total forces of attraction between clay particles and must be compensated for by a reduction in the total forces of repulsion. This is accomplished through an increase in the spacing between the particles, or a void ratio increase."

5.27 The average volume changes that did occur by curve-hopping during a CFS test are listed in TABLE V.2. These values divided by the appropriate differences in constant $\bar{\sigma}_1$ values are plotted against the logarithm of consolidation pressure in FIGURE V.2. The sodium modifications yield one line and the remainder of the soils tested provide another. The curves show the volume change that will occur due to the imposed $\bar{\sigma}_1$ difference given a consolidation pressure. TABLE V.1 lists theoretical 100% consolidation times which indicate that the sodium modifications are 60 to 800 times less permeable than the other soils tested. This is probably the reason for a separate curve for sodium. The calcium and magnesium modified soils and the remoulded and undisturbed clays have permeabilities in the same order of magnitude while the potassium soils are generally about 1/2 to 1/3 times as permeable.

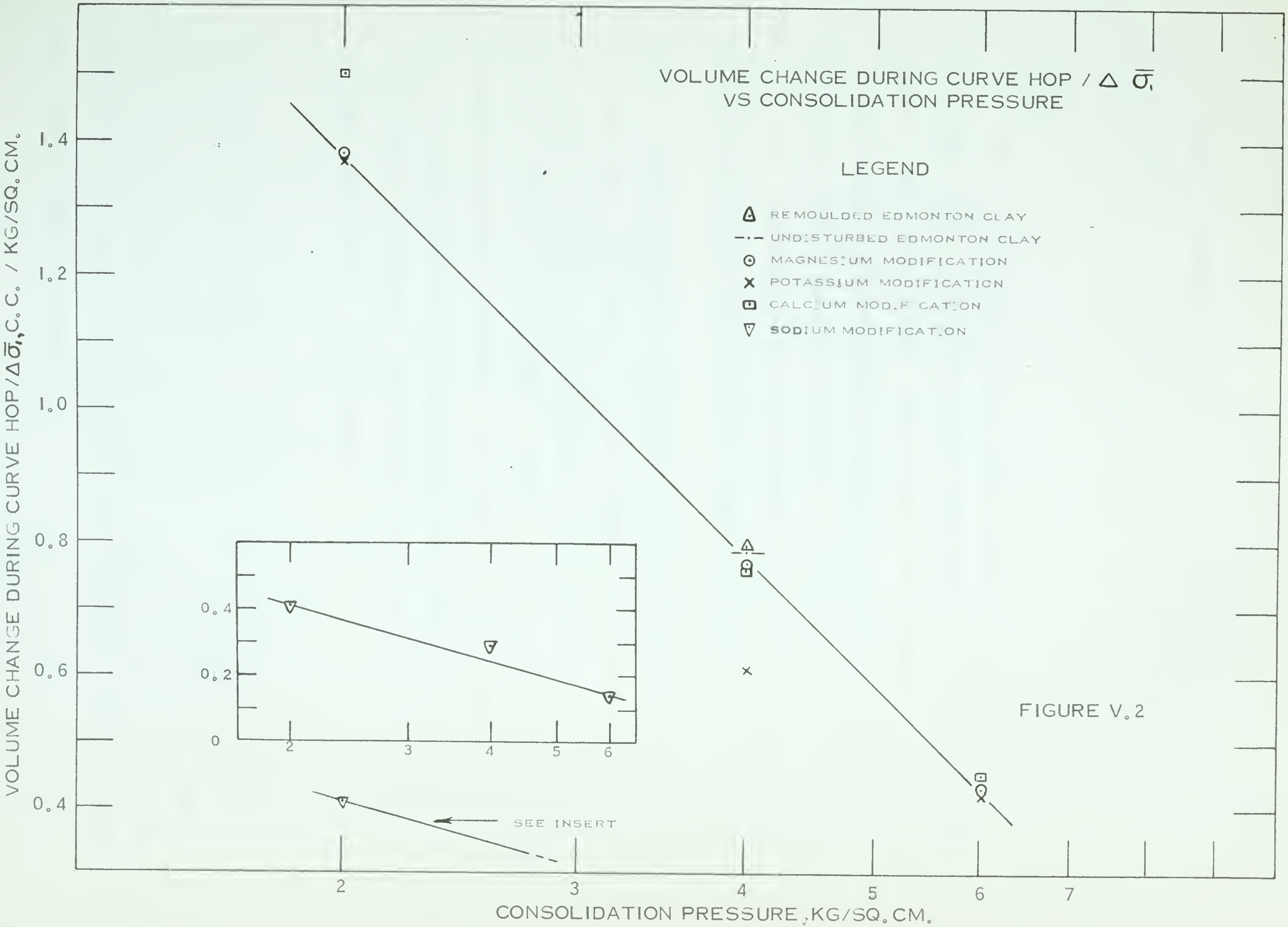

VOLUME CHANGE DURING CURVE HOP / $\Delta \bar{\sigma}_1$
VS CONSOLIDATION PRESSURE
LEGEND
REMOULDED EDMONTON CLAY
UNDISTURBED EDMONTON CLAY
MAGNESIUM MODIFICATION
POTASSIUM MODIFICATION
CALCIUM MODIFICATION
SODIUM MODIFICATION
VOLUME CHANGE DURING CURVE HOP/$\Delta\bar{\sigma}_1$, C.C. / KG/SQ.CM.
1.4
1.2
1.0
0.8
0.6
0.4
0.4
0.2
0
2
3
4
5
6
SEE INSERT
2
3
4
5
6
7
CONSOLIDATION PRESSURE, KG/SQ.CM.

FIGURE V.2

As the potassium modification tested at σ_c = 4.00 kg/sq. cm. was the most impermeable of the potassium specimens, the low value plotted in FIGURE V.2 may be attributed to this factor.

Summary

5.28 This chapter dealt with the data obtained and discussed the CFS triaxial test. A special triaxial consolidation test confirmed the suspicion that burette volume changes for all tests could be in error. A 3-minute correction was devised and applied to all data. Corrective measures for future tests were suggested. Calculation of volume change during a CFS test by three methods compared favorably. This change was shown to be dependent mainly upon the end-of-test strain.

5.29 A study of the most unfavorable combination of errors present in measurements made during a CFS test indicated the maximum error in the calculated cohesion to be approximately $\pm$ 0.1 kg/sq. cm. and in the angle of internal friction about $\pm$ 3 degrees. The rates of strain employed for this program were considered to be generally satisfactory from the point of view of pore pressure equalization and definition of curves. It was pointed out, however, that the rate of strain affects the shear resistance mobilized at a given strain.

5.30 The CFS test procedure involving volume changes was shown to be applicable to field problems. At a given consolidation pressure sodium modified soils were found to be the weakest and potassium the strongest of the remoulded specimens tested. Volume changes during curve-hops were shown to be predictable for the sodium soils on one hand and the remainder of the soils tested on the other.

CHAPTER VI

PRESENTATION OF RESULTS

Introduction

6.1 Cohesion, C_{ϵ} , and friction, $\emptyset_{\epsilon}$, are the mobilized components of the resistance of a soil to shear strain. The magnitude of each component varies with strain and the CFS test is the procedure used in this program to investigate this variation. It is emphasized that the CFS test is not intended as a means of determining the conventional Mohr-Coulomb shear strength parameters at failure. This chapter deals with the results of the CFS tests performed for this program and establishes general trends for the soil modifications used. Chapter VII is concerned more specifically with a physico-chemical interpretation of the results presented herein.

Computed Cohesion and Friction

6.2 Computations for cohesion (C_{ϵ}) and angle of internal friction ($\emptyset_{\epsilon}$) at any strain were made using the equations given in Paragraph 2.6. The data on which these calculations are based is taken from the deviator stress versus axial compressive strain curves presented in APPENDIX B. Two complete sets of data and calculations are given in APPENDIX C. Computations for cohesion and angle of internal friction were made at intervals varying between 0.5 and 1.2% axial strain, the points plotted and an estimated best fit line drawn through them. The

curves obtained in this manner are presented in FIGURES VI.1 to VI.12.

6.3 Each of FIGURES VI.1 to VI.6 contains plots of cohesion and angle of internal friction versus axial compressive strain for a given soil modification. For all soils tested except the sodium modification, cohesion is seen to peak at a relatively low strain whereas friction is mobilized at a much slower rate and reaches its maximum value at considerably higher strains. Cohesion for the sodium specimens reached its maximum value near the strain at failure with maximum friction generally occurring before this point. The strain at which the peak cohesion occurs for all soils tested appears to increase with increasing consolidation pressure. No trends are evident in this respect regarding maximum friction angle values.

6.4 Cohesion and friction curves for the remoulded Edmonton clay tested at a confining pressure of 6.10 kg/sq.cm. appear to be inconsistent with those from σ_3 = 2.0 and 4.0 kg/sq.cm. It is believed that they are incorrect due to poor technique. Referring to FIGURE B.1, APPENDIX B, it is seen that the deviator stress curves are not adequately defined in the first 7% axial strain. An error in the determination of one point due, for example, to non-equalization of pore pressure, could change the location of the initial portion of either stress-strain curve and thus significantly affect cohesion and friction computations. Assuming that this did occur, it could be shown that a larger difference in deviator stresses in the initial portions of the curves would result in strain-development of friction being similar to that for the other tests on this soil which would, in turn, flatten out the cohesion curve con-

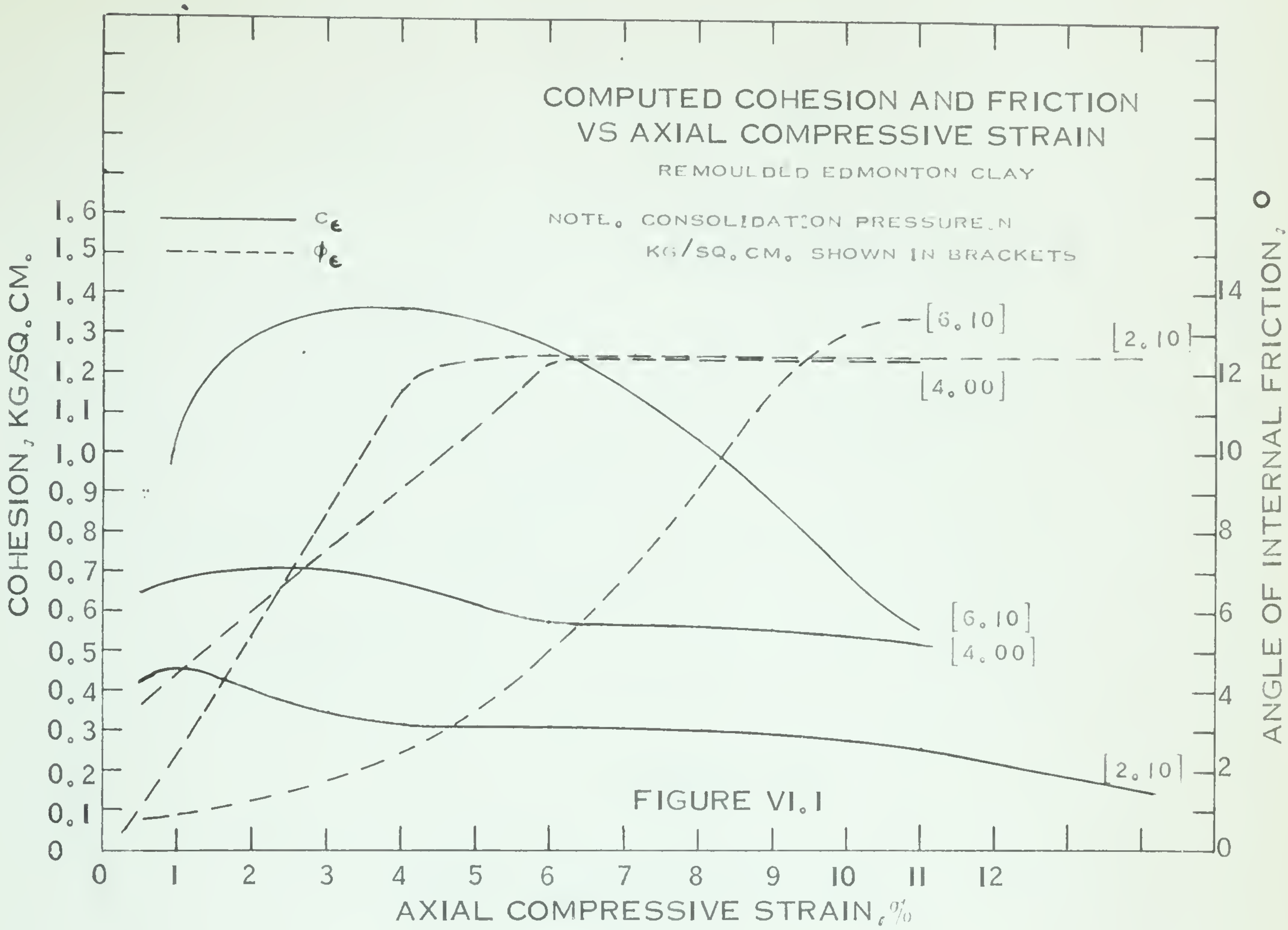


FIGURE VI.1

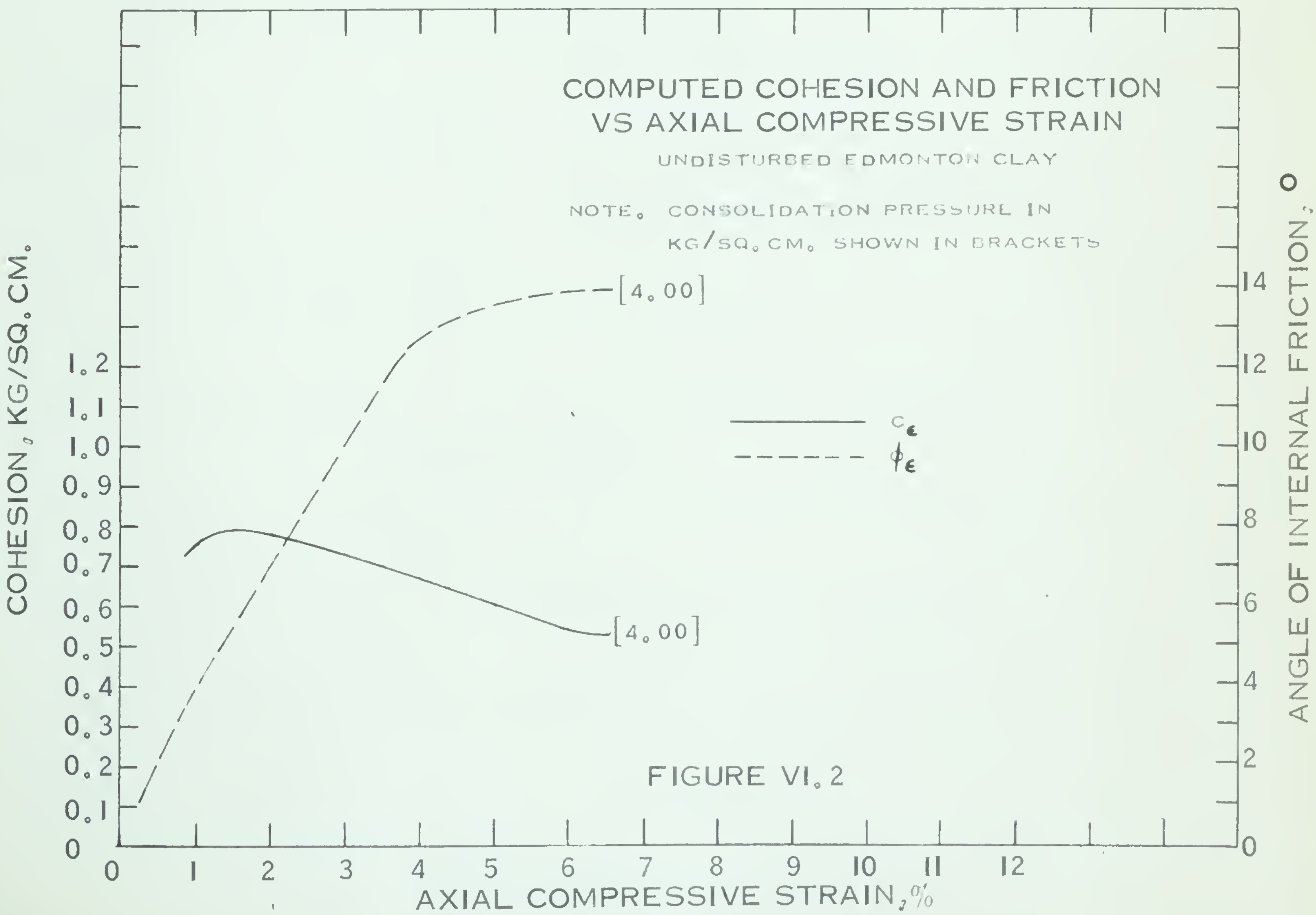


FIGURE VI.2

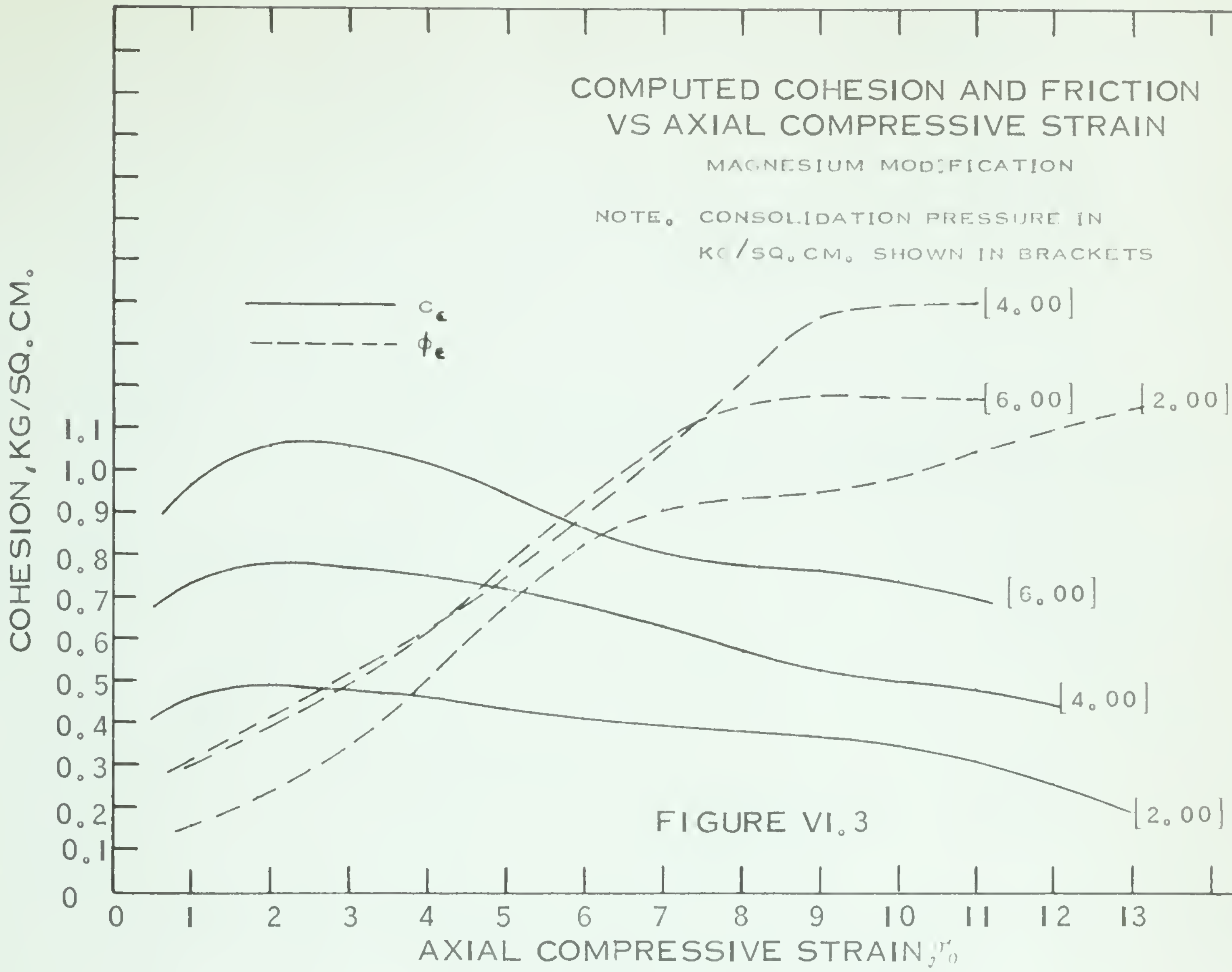


FIGURE VI.3

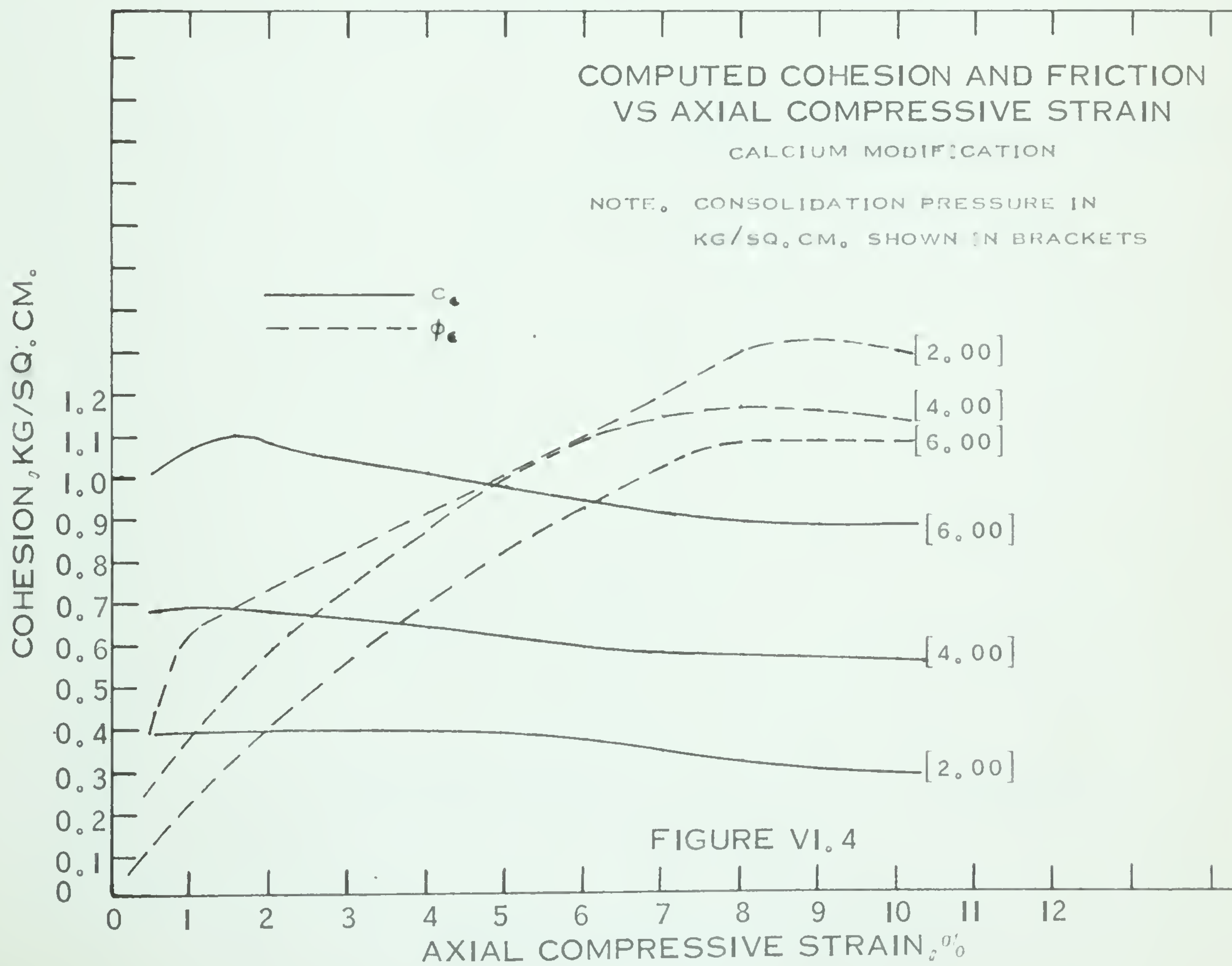


FIGURE VI.4

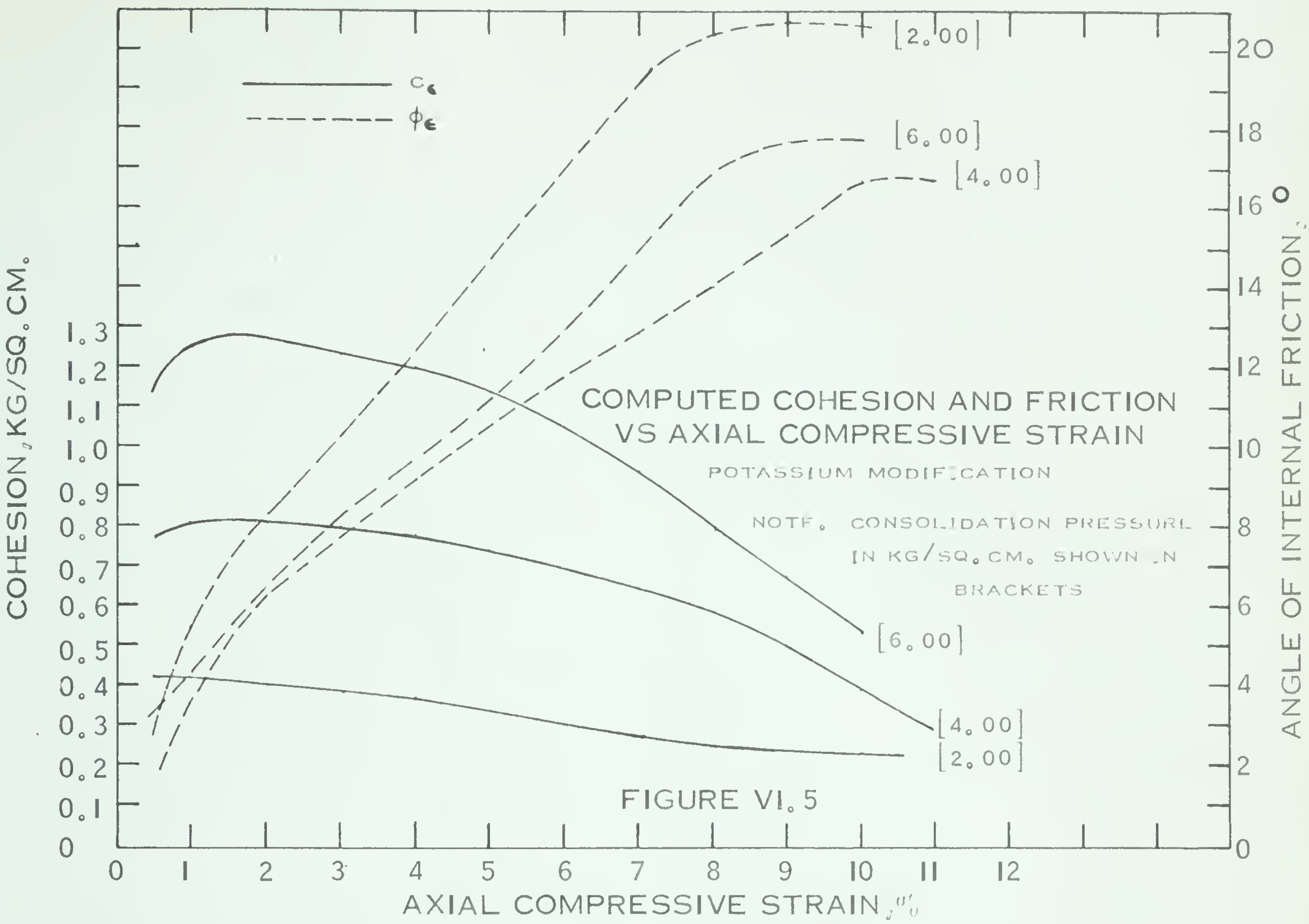


FIGURE VI.5

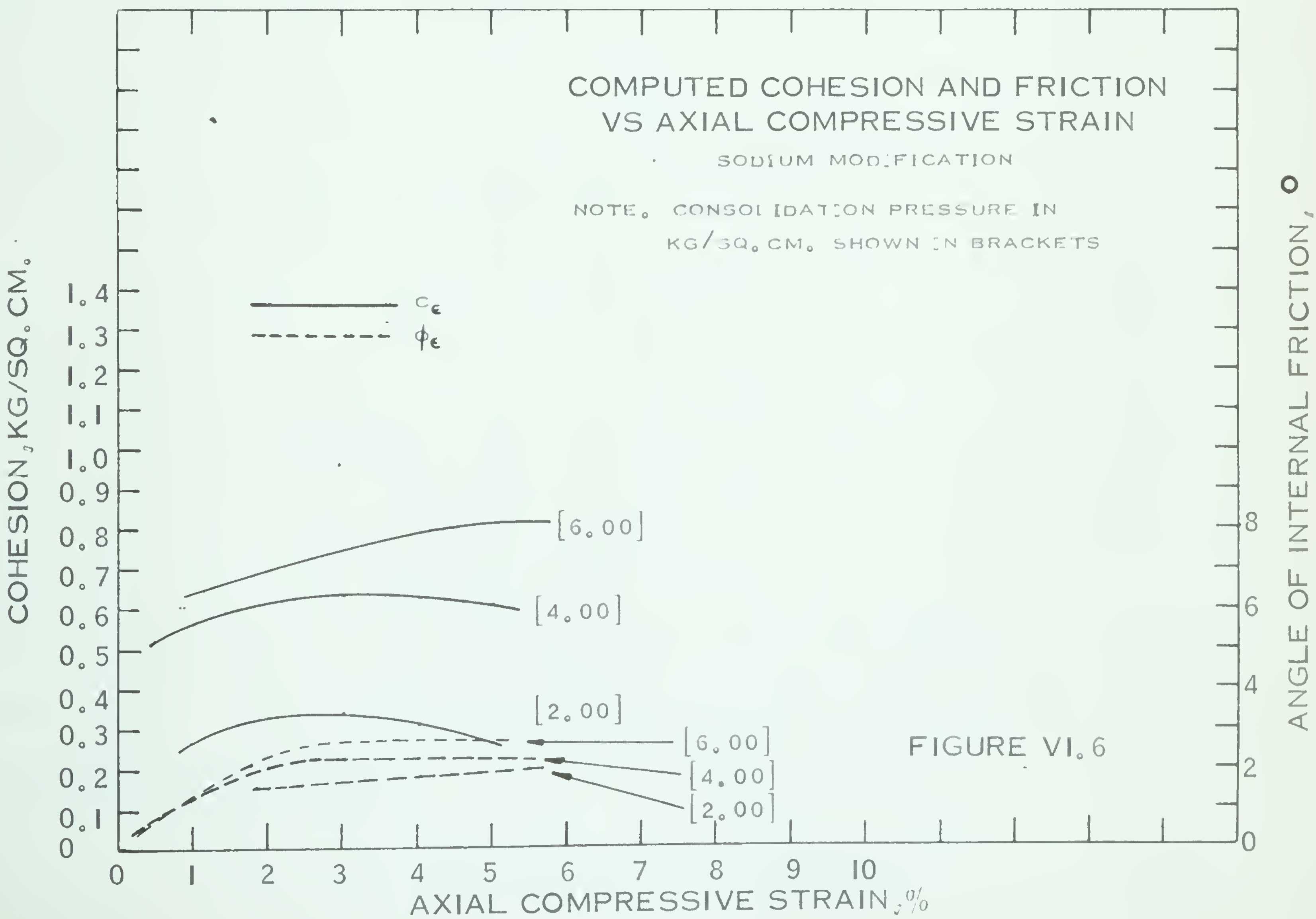


FIGURE VI.6

siderably making it more consistent with the other curves shown.

6.5 For a given soil modification, the regular displacement of the cohesion curves suggests the dependency of cohesion on the consolidation pressure. Strain does not appear to disrupt high-strain displacements as these curves maintain approximately their relative positions throughout the progress of the tests. Friction curves for a given soil are similar both in shape and position. Differences in maximum values attained do not vary consistently with consolidation pressure. The writer feels that the discrepancies shown are mainly due to errors in measurements as described in Paragraph 5.5. Differences in angle of internal friction values at any strain are within the ordinary range of errors that might occur. The peak cohesion and maximum angle of internal friction computed for each test performed are listed in TABLE V.1. Averaging the maximum friction angle values obtained and comparing these yields the following order: potassium (18°34'), undisturbed clay (14°25'), magnesium (13°36'), remoulded clay (13°22'), calcium (11°57') and sodium (2°20').

6.6 In order to illustrate more effectively the relation between consolidation pressure, cohesion and angle of internal friction FIGURES VI.7 to VI.12 are presented. These are plots of angle of internal friction versus strain and cohesion versus strain at consolidation pressures of 2, 4 and 6 kg/sq.cm. Each diagram contains applicable curves from all soils tested with the exception of the results obtained from Test No. 2. The curves from this test are erroneous (Paragraph 5.17) and therefore misleading.

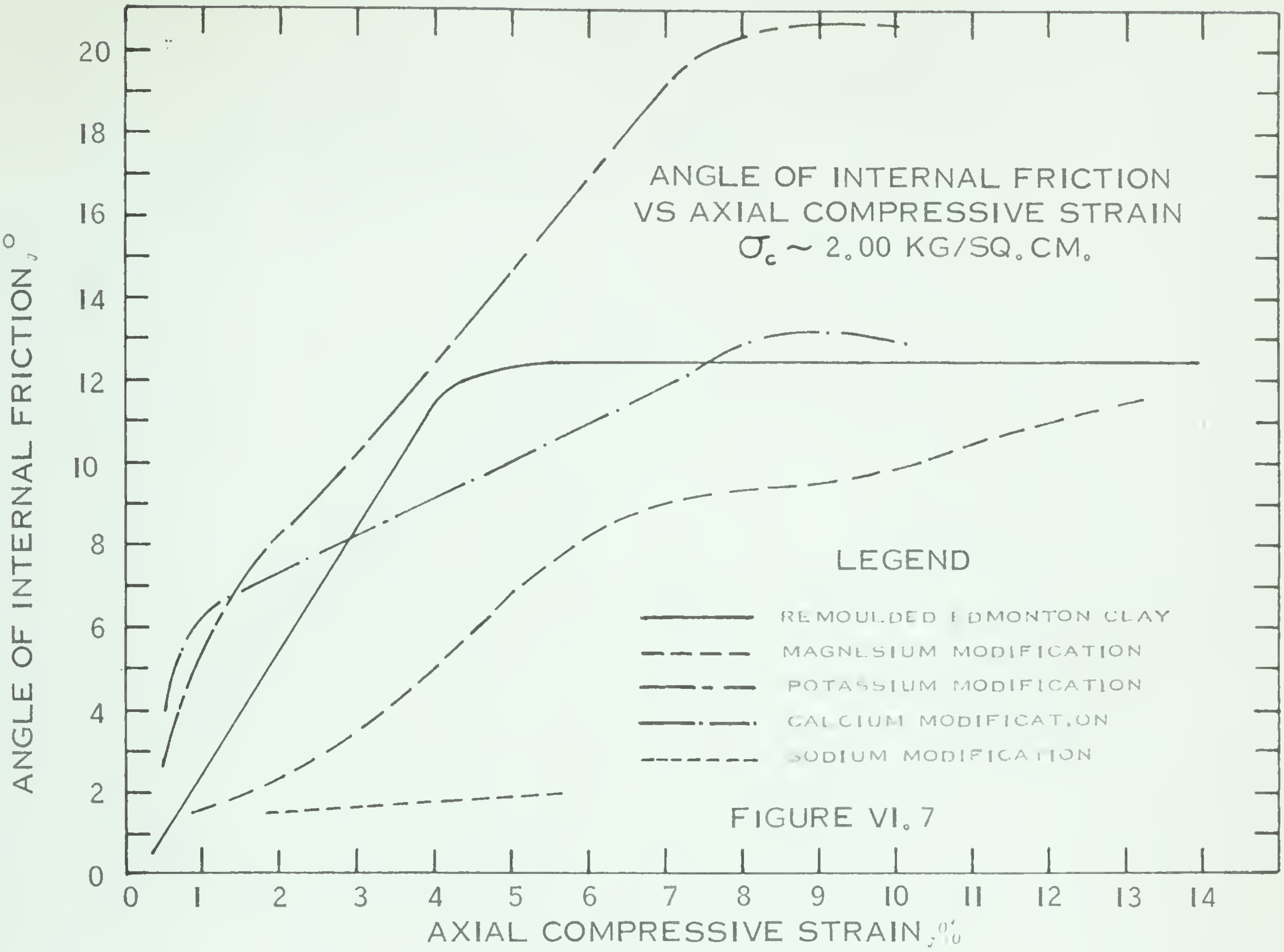


FIGURE VI.7

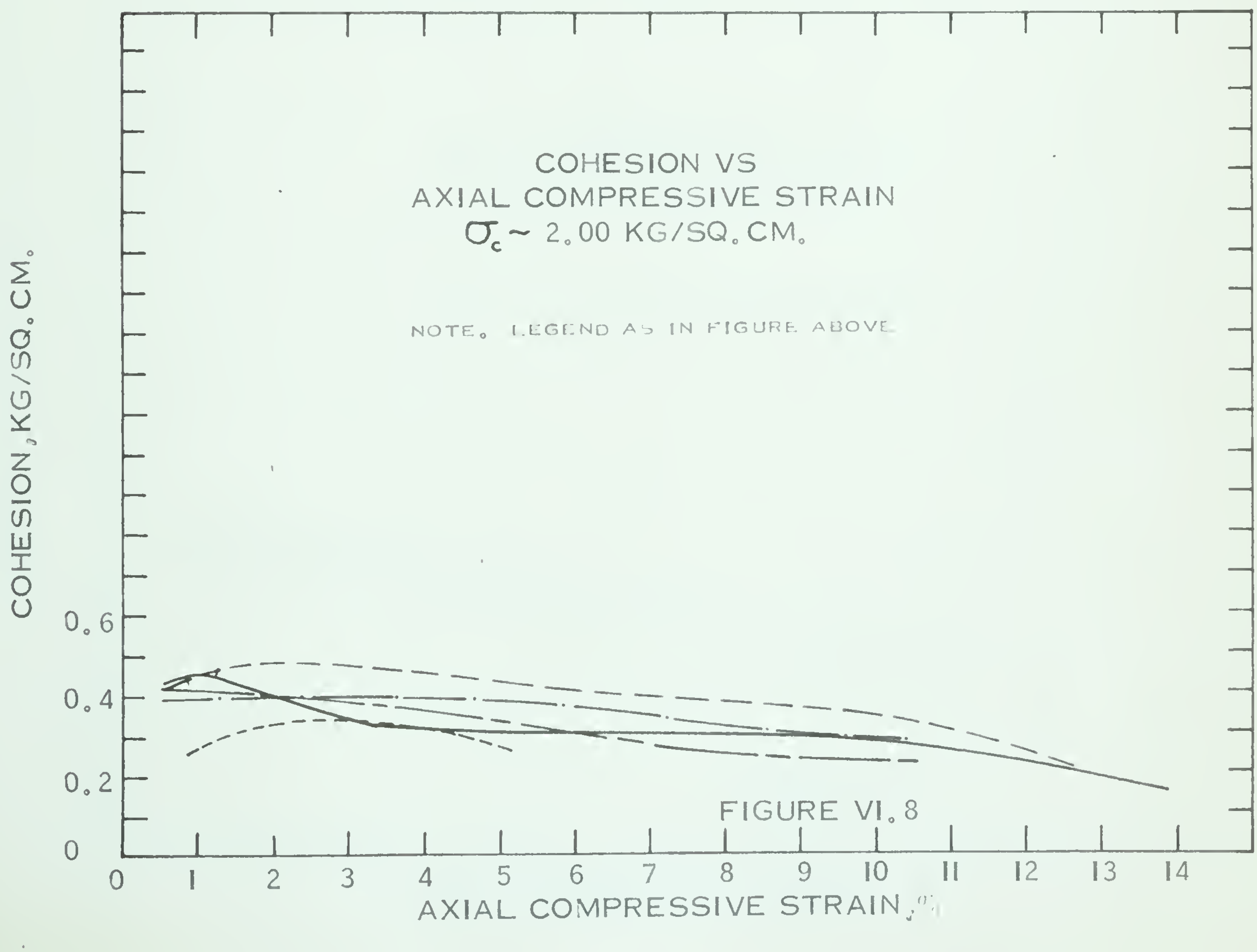


FIGURE VI.8

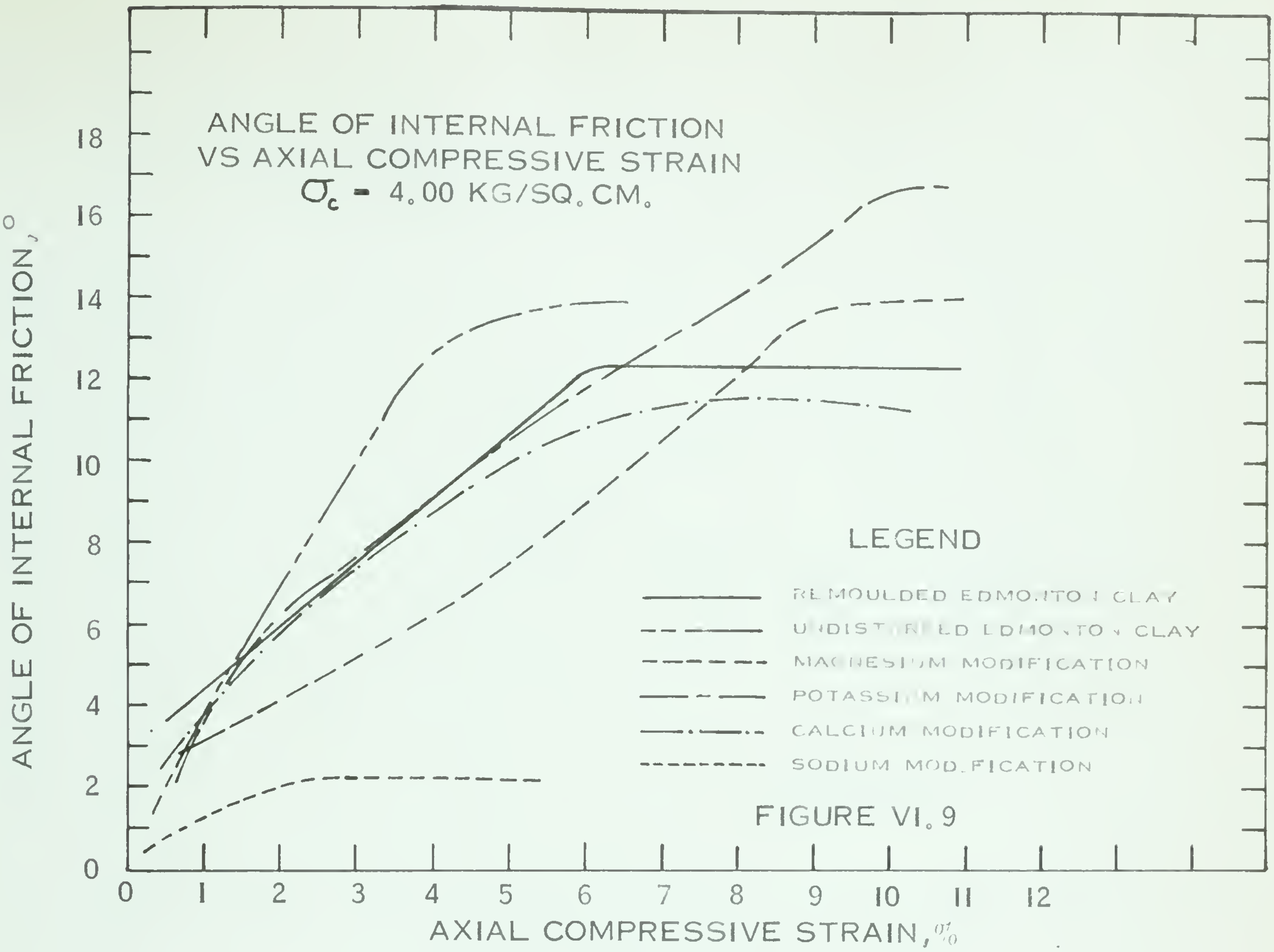


FIGURE VI.9

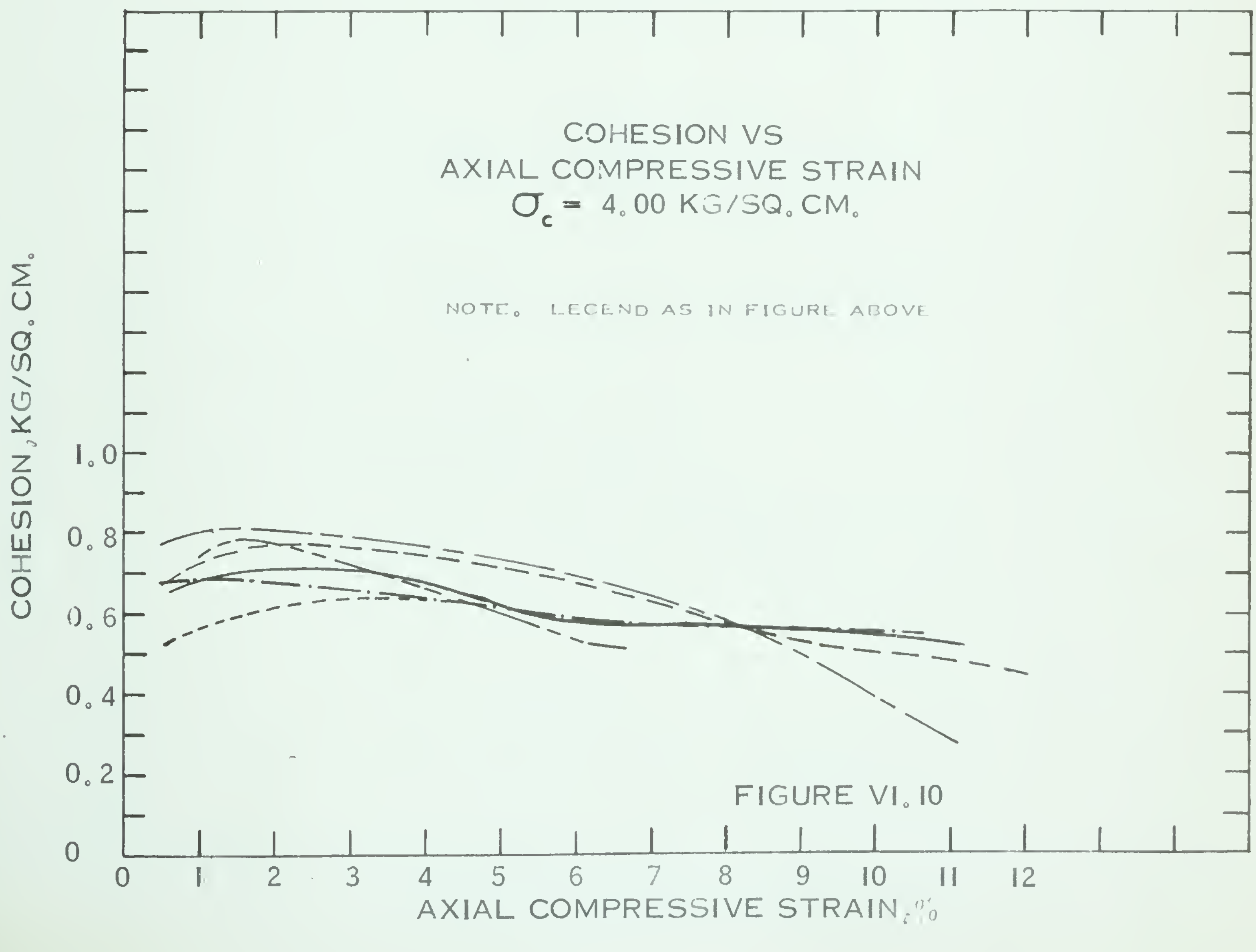


FIGURE VI.10

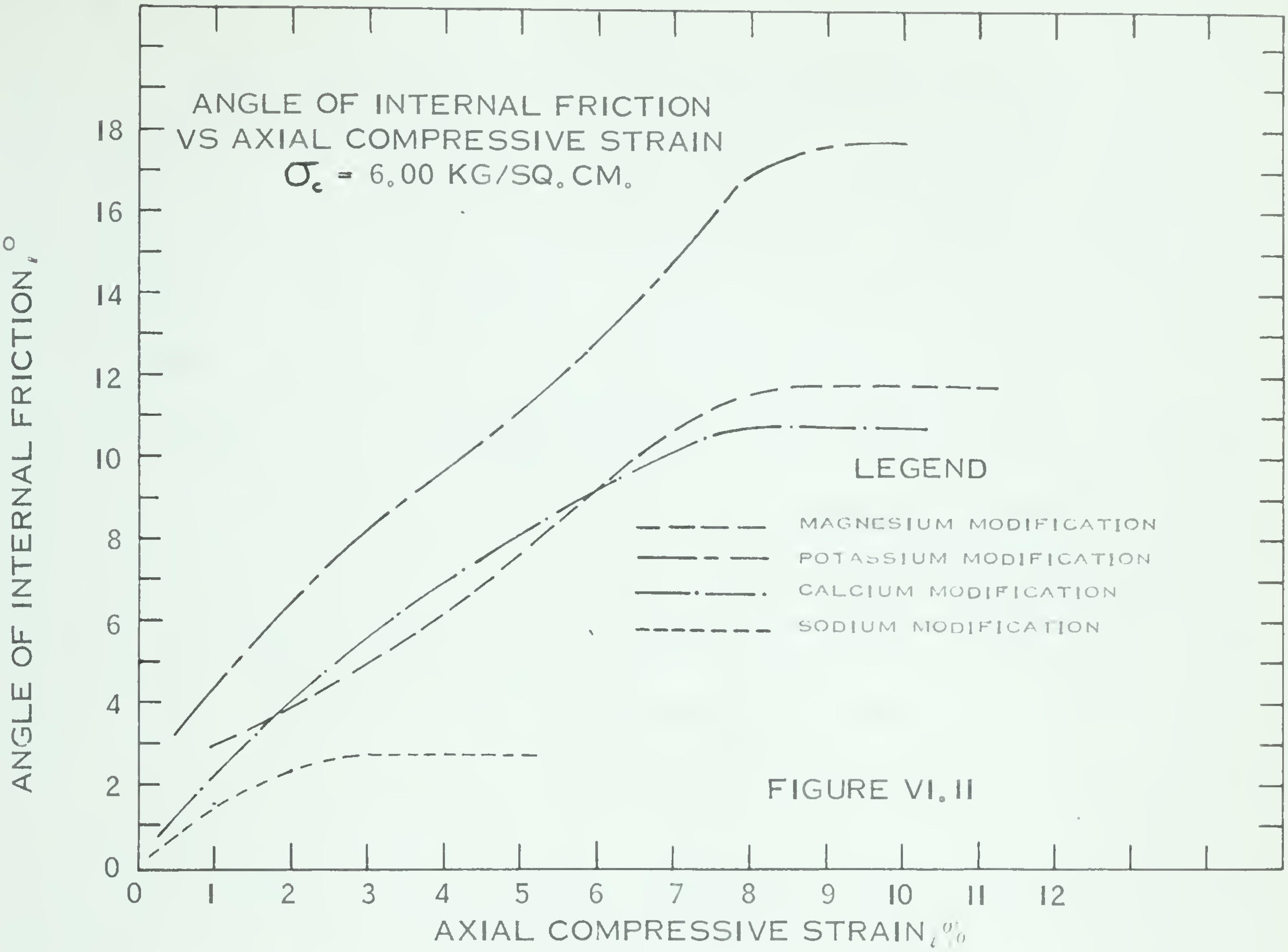


FIGURE VI.11

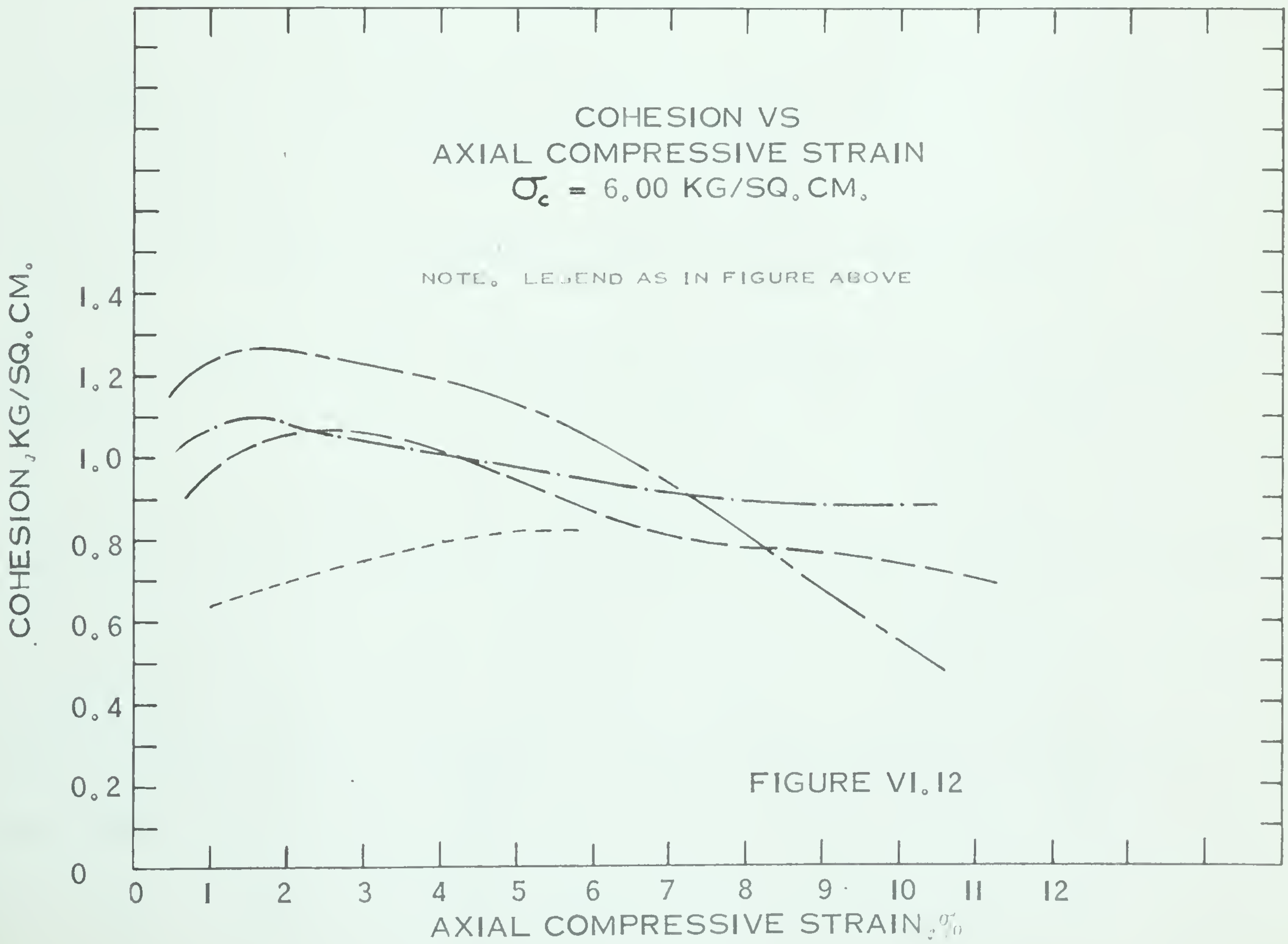


FIGURE VI.12

6.7 Considering, first, the cohesion versus axial compressive strain plots, it is seen that a distinct band is formed which includes all soils tested at a given consolidation pressure. This band is best defined for the tests at which σ_c = 2.00 and 4.00 kg/sq.cm. The mobilized cohesion appears to be independent of the adsorbed cation and simply dependent on consolidation pressure. The only apparent exception to this is the potassium cohesion curves at strains in excess of 8 to 9%. The relatively rapid decrease in cohesion here is due to the rapid decrease in strength described in Paragraph 5.24. This, in turn, was attributed to the brittle nature of potassium modified soils.

6.8 Angle of internal friction versus strain plots do not exhibit any definite relation to consolidation pressure. This is illustrated in FIGURE VI.13(a). Individual curves do, however, appear to be related to the type of cation adsorbed. The order of peak friction angle values attained at σ_c = 4.00 and 6.00 kg/sq.cm. follows exactly the order of average values presented at the end of Paragraph 6.5. At σ_c = 2.00 kg/sq.cm. the calcium and magnesium peak values are interchanged.

6.9 FIGURE VI.13(b) is another way of presenting the data shown in FIGURES VI.8, VI.10 and VI.12. Peak cohesion for the various modifications is plotted against consolidation pressure. As expected, a band is obtained which illustrated the dependency of cohesion on consolidation pressure and the lack of influence of the type of adsorbed cation. The dashed lines illustrate this relation at 10% axial compressive strain. The cross-hatched area encompasses values of peak cohesion obtained by Schmertmann and Osterberg (1960) from their program on soils whose

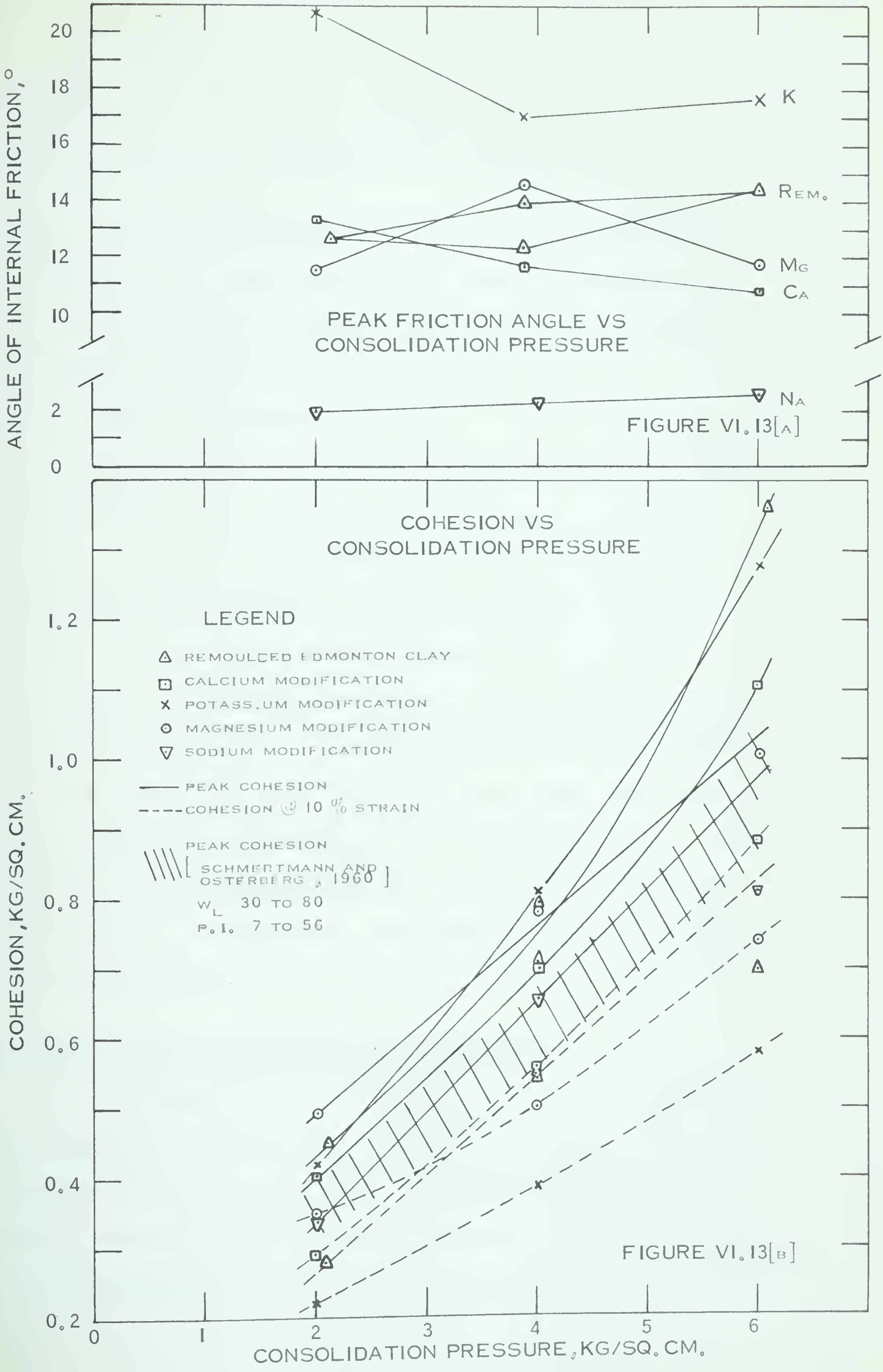


FIGURE VI. 13[A]

FIGURE VI. 13[B]

liquid limits varied from 30 to 80 and plasticity indices ranged from 7 to 56. This band is parallel to that obtained from this program.

6.10 As the plasticity characteristics of the soils used for this program are at the upper limits of the ranges investigated by Schmertmann and the band from this program is just above that determined by Schmertmann, it appears that the peak cohesion may also be a function of the plasticity characteristics of a soil which in turn, are influenced by the grain size distribution, the type of clay minerals present and the adsorbed cations. The effect of these factors on the peak cohesion determined for the soils of this program cannot be pursued further here for three main reasons. Firstly, the range of plasticity of the soils tested is not large enough to make a proper analysis. Secondly, more precise results than those presented herein would be required to make an adequate analysis. Thirdly, the quantity of soil prepared for this program did not allow classification tests other than specific gravity analyses to be performed on the soil modifications. Time also necessarily limits this program and thus the effects of plasticity characteristics on cohesion as determined by the CFS test are suggested for future research.

<u>Cohesion, Compressive Strength and Consolidation</u>

6.11 FIGURES VI.14 to VI.18 present curves of the logarithm of the compressive strength, the consolidation pressure and twice the peak cohesion versus moisture content. Consolidation pressure and twice the peak cohesion are plotted against the moisture content after consolidation as determined from corrected burette volume change measurements

LEGEND TO FIGURES V.14-18
SUMMARY OF CFS TEST RESULTS

——Δ— ——	Peak Cohesion x 2 vs Moisture Content After Triaxial Consolidation
- - - - x - - -	Compressive Strength vs Final Moisture Content When $\bar{\sigma}_1 \sim \sigma_c$
— — + — —	Compressive Strength vs Final Moisture Content When $\bar{\sigma}_1 < \sigma_c$
——⊙——	Triaxial Consolidation Pressure vs Moisture Content After Consolidation (FIGURE V.15-18)
——○——	Triaxial Consolidation Pressure vs Moisture Content (From one Consolidation Test) (FIGURE V.14)
⊡	Consolidated Undrained Test Result (FIGURE V.14)

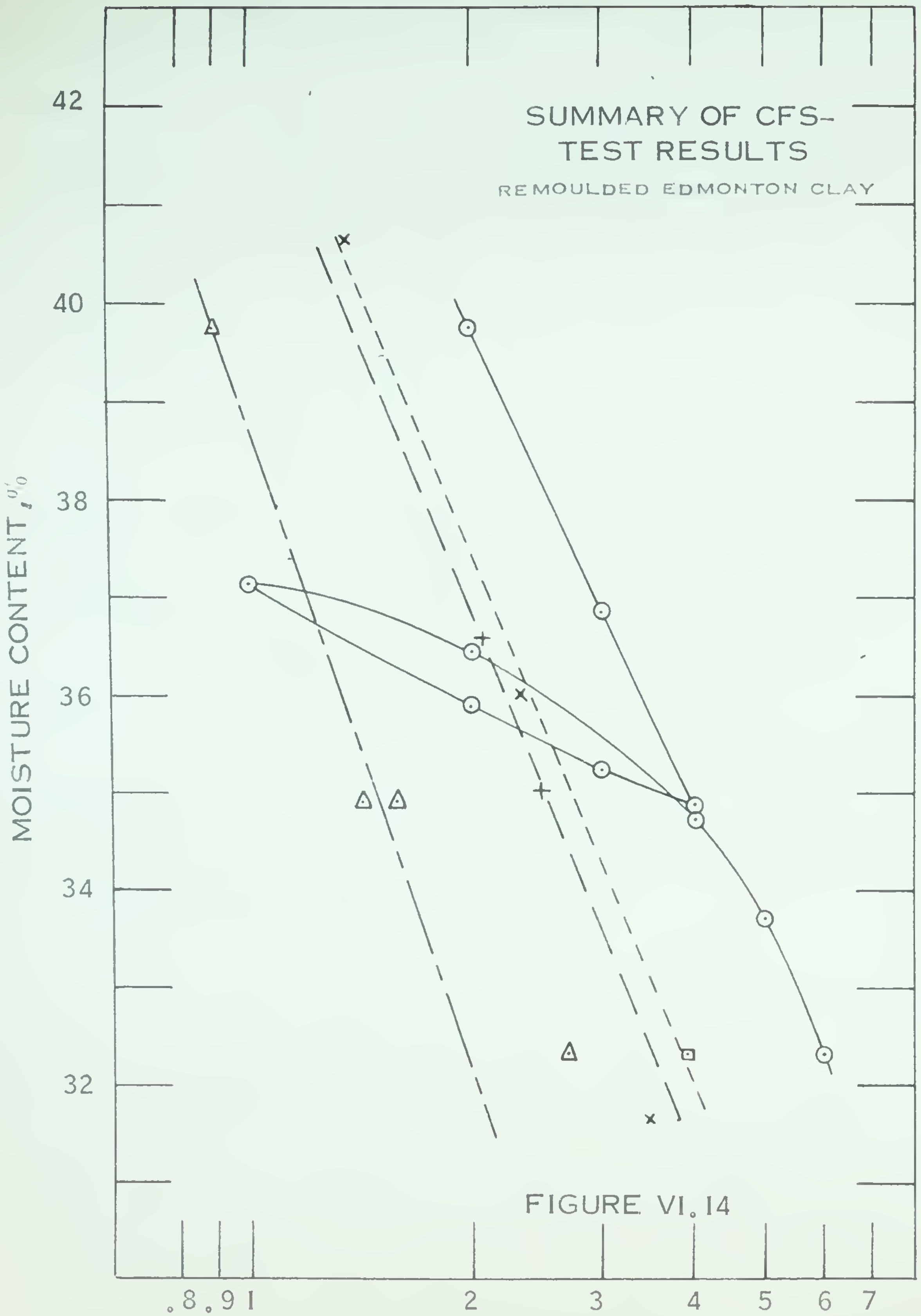


FIGURE VI.14

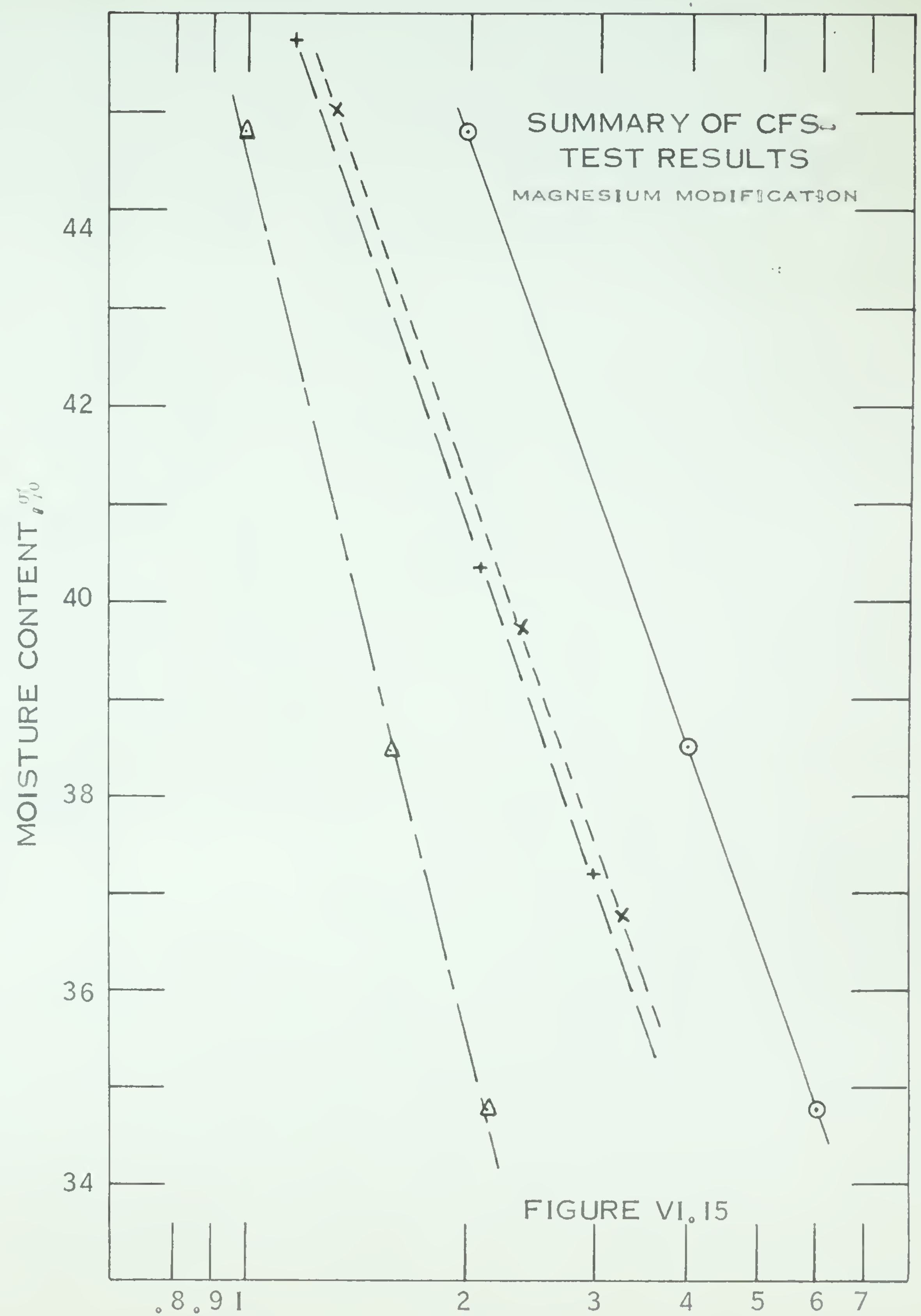


FIGURE VI.15

PEAK COHESION, COMPRESSIVE STRENGTH, AND CONSOLIDATION PRESSURE, KG/SQ.CM.

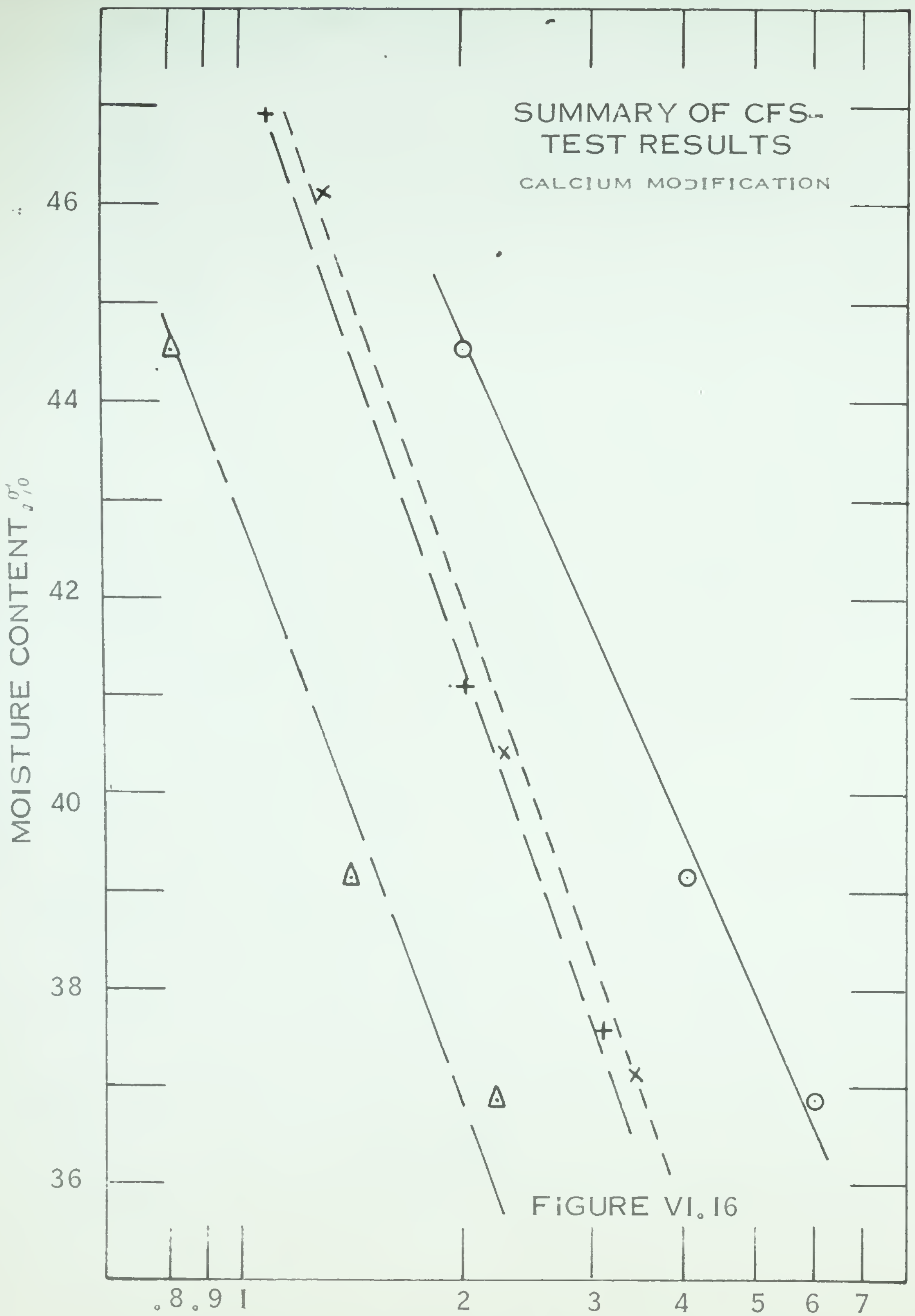


FIGURE VI.16

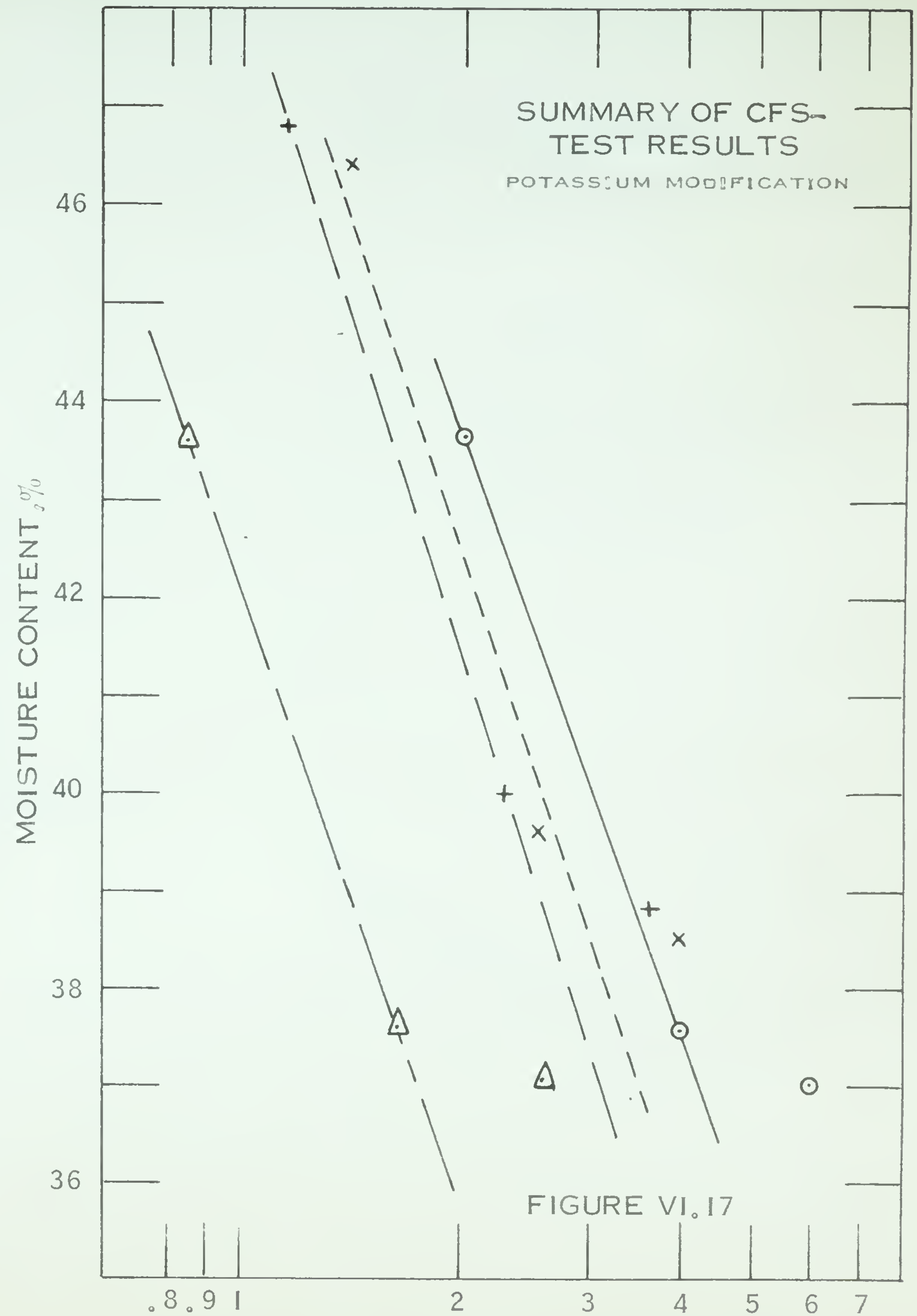


FIGURE VI.17

PEAK COHESION, COMPRESSIVE STRENGTH, AND CONSOLIDATION PRESSURE, KG/SQ. CM.

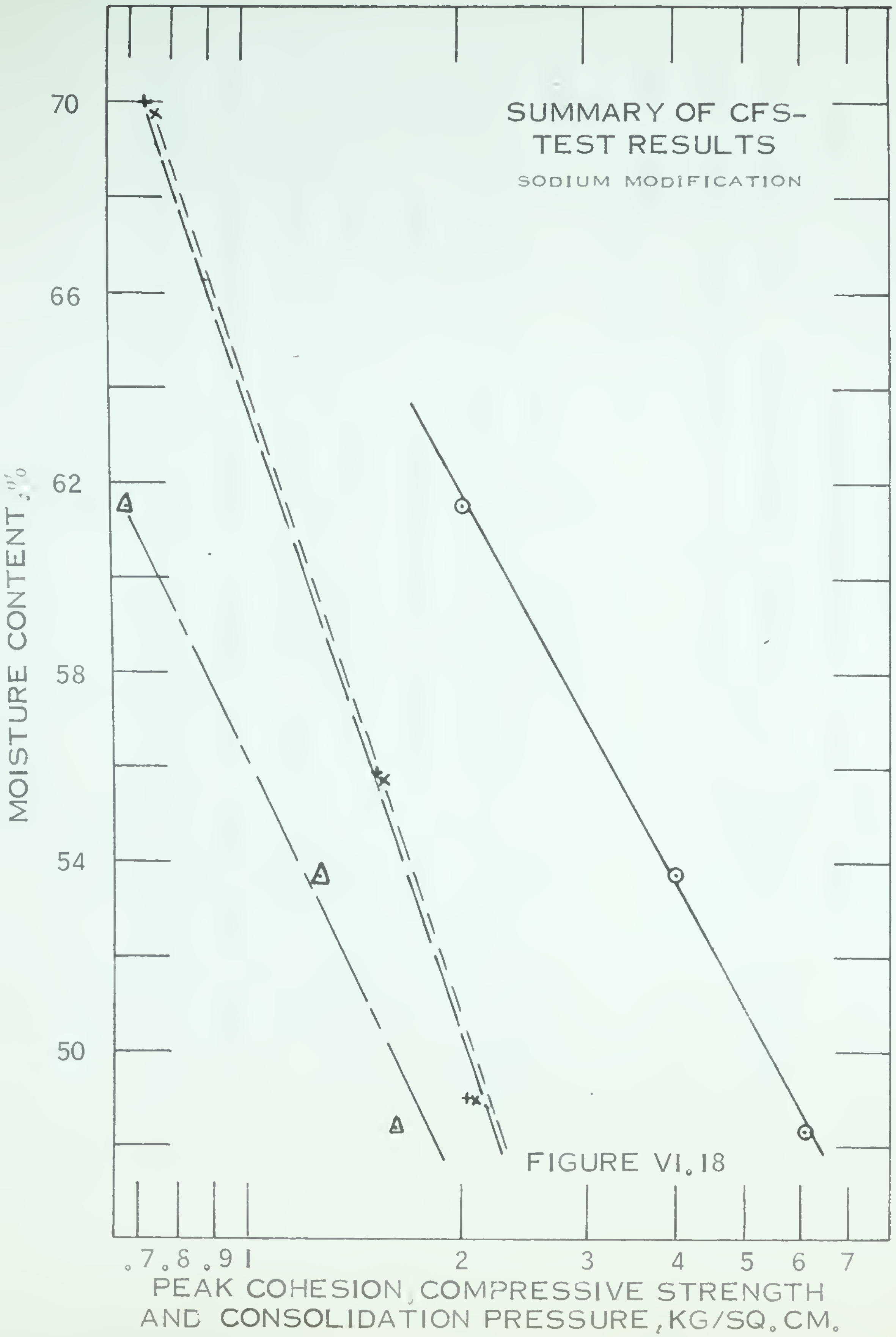


FIGURE VI.18

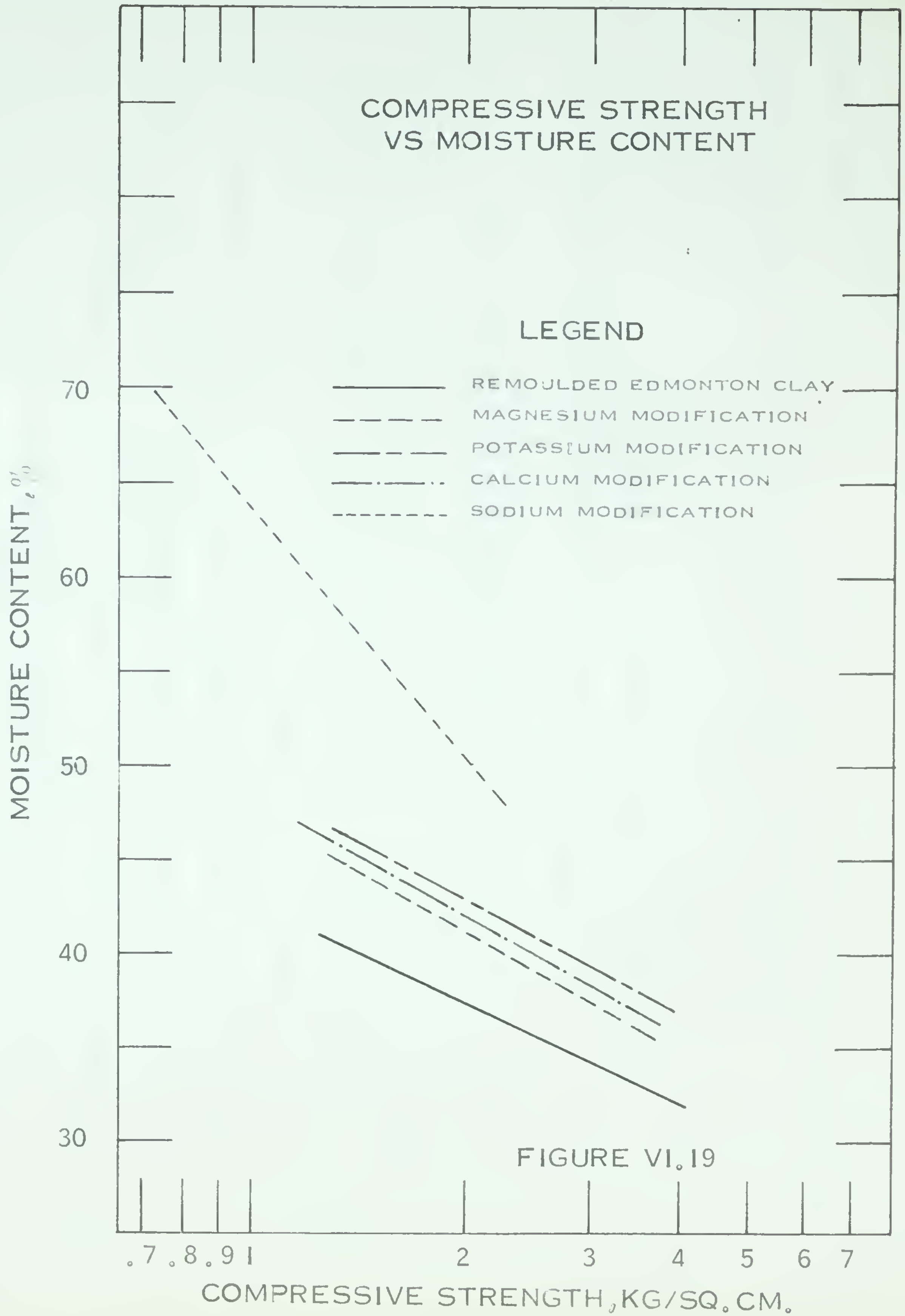


FIGURE VI.19

described previously. The resulting curves are well defined and thus the method of correction (Paragraph 5.3) appears to be successful. One exception to this is the result obtained from the potassium modification tested at σ_c = 6.00 kg/sq.cm. Calculated moisture content after consolidation is high and thus the correction was apparently too great.

6.12 The generally acknowledged linear relationship between the logarithm of consolidation pressure and moisture content (for saturated systems) is reaffirmed. Peak cohesion is seen to exhibit a similar relation. If, in fact, cohesion is a function of consolidation pressure then this relationship must exist because, for a given saturated soil system, void ratio and thus moisture content, is determined by the consolidation pressure. This does not mean, however, that for a given consolidation pressure, and thus peak cohesion, there is a fixed moisture content for all soil systems. On the contrary, the moisture content at a given consolidation pressure varies with the nature of the soil system. An equivalent cohesion, for example, is provided by the sodium modifications at a much higher moisture content than the other soils tested.

6.13 It has been generally accepted that a linear relationship exists between void ratio, or moisture content for saturated systems, and the logarithm of the compressive strength of a given clay. As long as there are no structural disturbances, this relationship appears to be independent of all other variables (Taylor, 1948 p. 370). This curve is also generally parallel to the virgin compression branch of the consolidation curve. The relationship shown in FIGURES VI.14 to

VI.18 essentially fall in line with the preceding statements. It is seen, however, that two strength lines are obtained from the results of a series of CFS tests on a given soil. The line nearer the consolidation curve represents compressive strengths for $\bar{\sigma}_1 \sim \sigma_c$ and the line immediately to the left of this strength line represents compressive strength for $\bar{\sigma}_1 < \sigma_c$. The former may be considered a compressive strength curve for the condition of normal consolidation whereas the latter may be thought of as the curve for slight over consolidation.

6.14 This separation of the compressive strength curves does not, at first glance, appear consistent with the idea that soil specimens of a given type at similar structures should yield a unique curve. Schmertmann and Osterberg, quoted in Paragraph 5.26, demonstrated that slightly different structures must result from changes in $\bar{\sigma}_1$. Taylor, referred to in the previous paragraph, indicated that the location of the compressive strength - moisture content curve could be dependent on the structure of the soil tested. It is not surprising, then, that two different curves were found from the results of the CFS tests.

6.15 Simons (1960), who performed drained and undrained triaxial strength tests on normally consolidated and over-consolidated specimens of an Oslo clay, also shows a separation of compressive strength-moisture content curves for these two conditions. This work lends weight to the argument that separation of the compressive strength curves is due to structural differences within a given type of soil.

6.16 Consideration of the composite test results presented in FIGURE D.1, APPENDIX D, also tends to confirm the preceding arguments. Compressive strength versus moisture content curves from normally consolidated specimens are adjacent to curves from over-consolidated specimens. The distance between the two curves may be a measure of the structural differences resulting from over consolidation. It is seen that this distance is smaller, in FIGURE D.1, for the CFS test results than the distance between the curves where a relatively large over-consolidation ratio has been investigated (OCR from 3 to 8).

6.17 One additional factor must be mentioned here which may affect the displacement of the compressive strength curves obtained from the CFS test. At any given strain, a jump from the high $\bar{\sigma}_1$ curve to the low $\bar{\sigma}_1$ curve is associated with a volume change resulting from forcing a small quantity of water into the specimen. Taylor (1948, P. 382, footnote) states that "resistance to volume increase and probably other phenomena have some effect on the position of these lines." The phrase "these lines" refers to lines representing stresses at failure on a void ratio-logarithm of stresses plot and thus includes compressive strength lines. Resistance of the specimen to volume increase under axial stress probably does contribute to the displacement of the compressive strength lines from the CFS tests. A further analysis of this factor is, however, not possible and thus must be accepted simply as a possible contributing factor.

6.18 FIGURE VI.19 contains the compressive strength lines for the remoulded soils tested when $\bar{\sigma}_1 \sim \sigma_c$. With the exception of

the sodium soil, all strength lines are essentially parallel. The results of strength test performed by Rosengvist (1955) on homionic modifications of a given clay illustrated that different cations simply shifted the compressive strength line up or down with the lines remaining parallel, independent of the size of the cation. The work of Locker (1963) showed that the position and slope of this line was also a function of the salt content of the pore water, particularly for sodium modified soils.

6.19 The calcium, magnesium and potassium modifications of this program had approximately the same salt concentrations in the pore water and thus the positions of these strength lines on this plot may be considered a function of the type of cation adsorbed. Locker's work showed that the position of the strength line for the homionic calcium clay moved downward with increasing salt content. As the natural remoulded clay tested in the program of this thesis was predominantly calcium saturated and had a high salt content, its strength line on FIGURE VI.19 is in the expected position relative to that of the calcium soil strength line.

6.20 Locker also pointed out that the slope of the strength line for a sodium modified clay flattened with increasing salt content. The strength line for the sodium modified soil of this program fits extremely well into the plot of moisture content versus compressive strength presented by Locker (1963, p. 44). The sodium soil used for the program of this thesis had the lowest salt content of all soils prepared. If the salt content were increased to correspond to that found in the other

modified soils of this program, the strength line would flatten and, as suggested by Locker's plot, become parallel to the other lines shown.

Mohr Diagrams

6.21 The Mohr Diagram is usually used to represent stress conditions at failure within a soil mass. The Mohr failure envelope is the line representing the locus of points showing stress conditions on the failure plane. The Coulomb-Hvorslev failure equation, which forms the basis for calculation of cohesion and friction at any strain during the CFS test, is simply the equation of a Mohr failure envelope. The line of reasoning followed by Schmertmann (1960) to justify the use of this equation at any strain reads as follows:

> "Since a Mohr's circle represents any two-dimensional stress condition of equilibrium, it can be used at any strain value before failure is reached with the same validity as at the failure condition conventionally chosen. The envelope formed by the points of tangency to the Mohr circles representing intergranular stresses at a point for a given value of strain can be used to find the cohesion and friction at that strain. If the Mohr circles for different intergranular stresses are all found at the same soil structure, the cohesion and angle of internal friction are obtained."

6.22 Schmertmann's reasoning leading to the determination of cohesion and friction at any strain must be clearly kept in mind as methods of interpretation of data presented in the following paragraphs differ in certain respects from this approach. Methods of interpretation of shear strength data outlined in Chapter II will be referred to and the difference between these and the CFS test analysis noted.

6.23 FIGURES VI.20 to VI.25 are Mohr Diagrams which contain failure

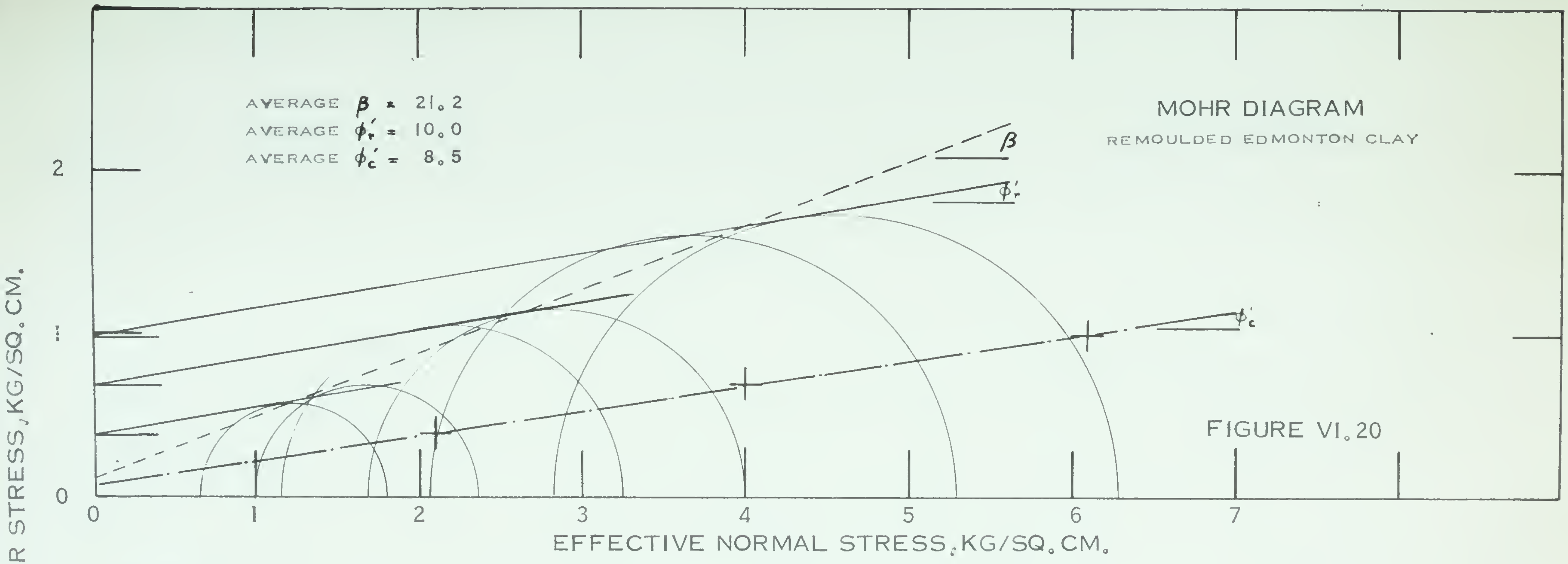


FIGURE VI.20

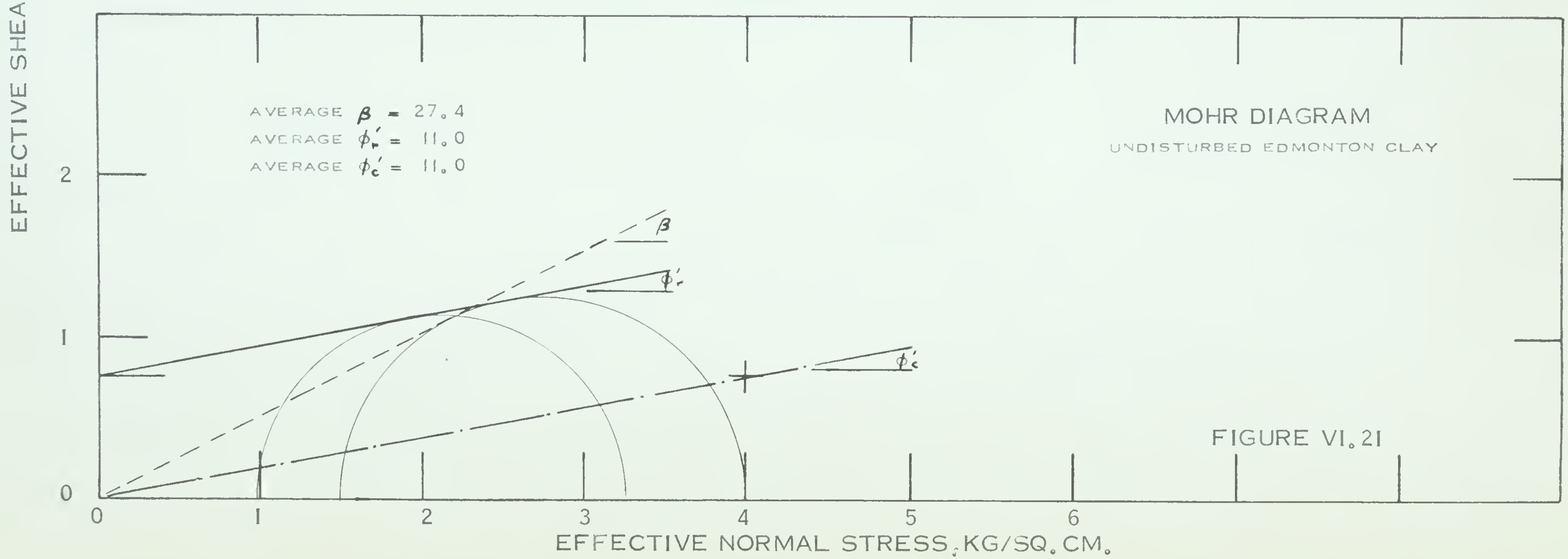


FIGURE VI.21

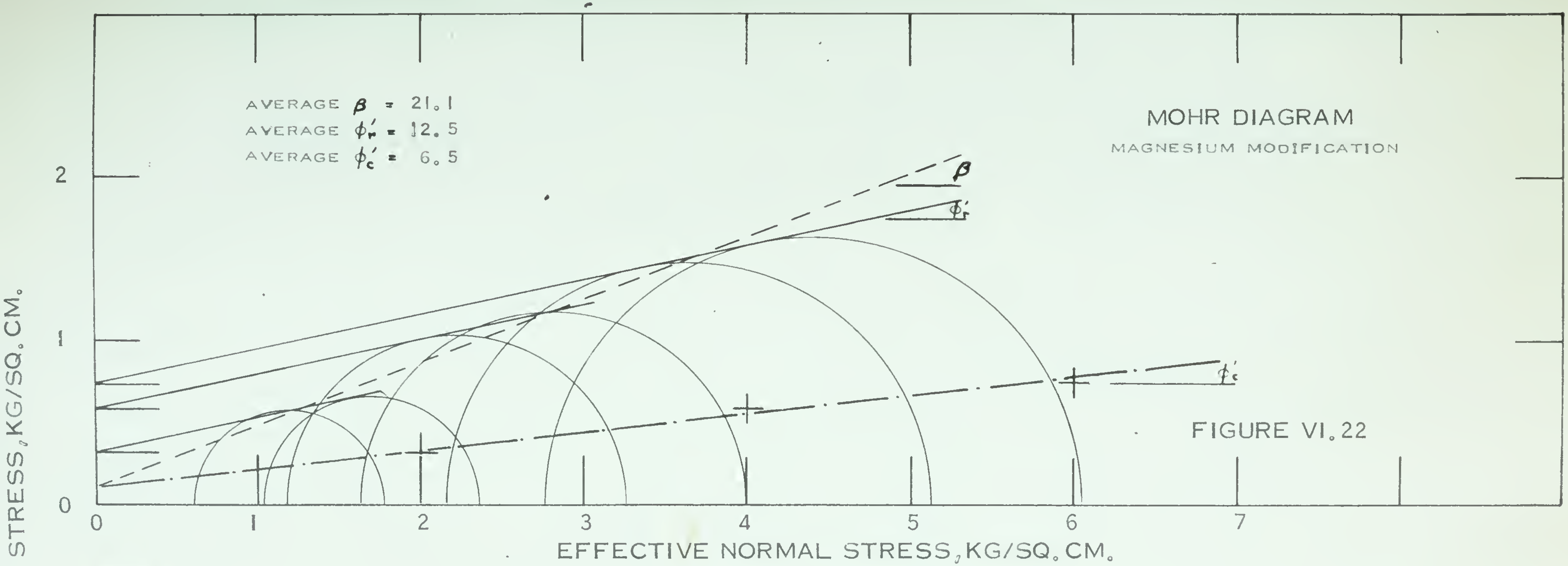


FIGURE VI.22

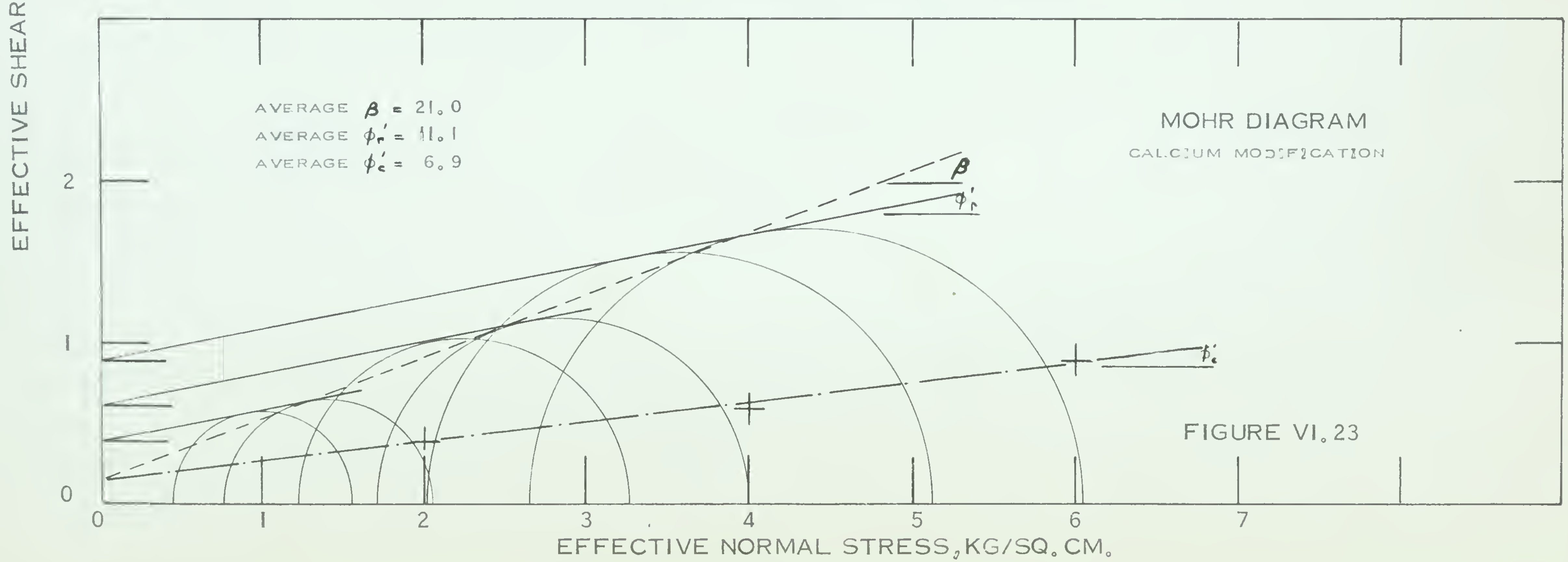


FIGURE VI.23

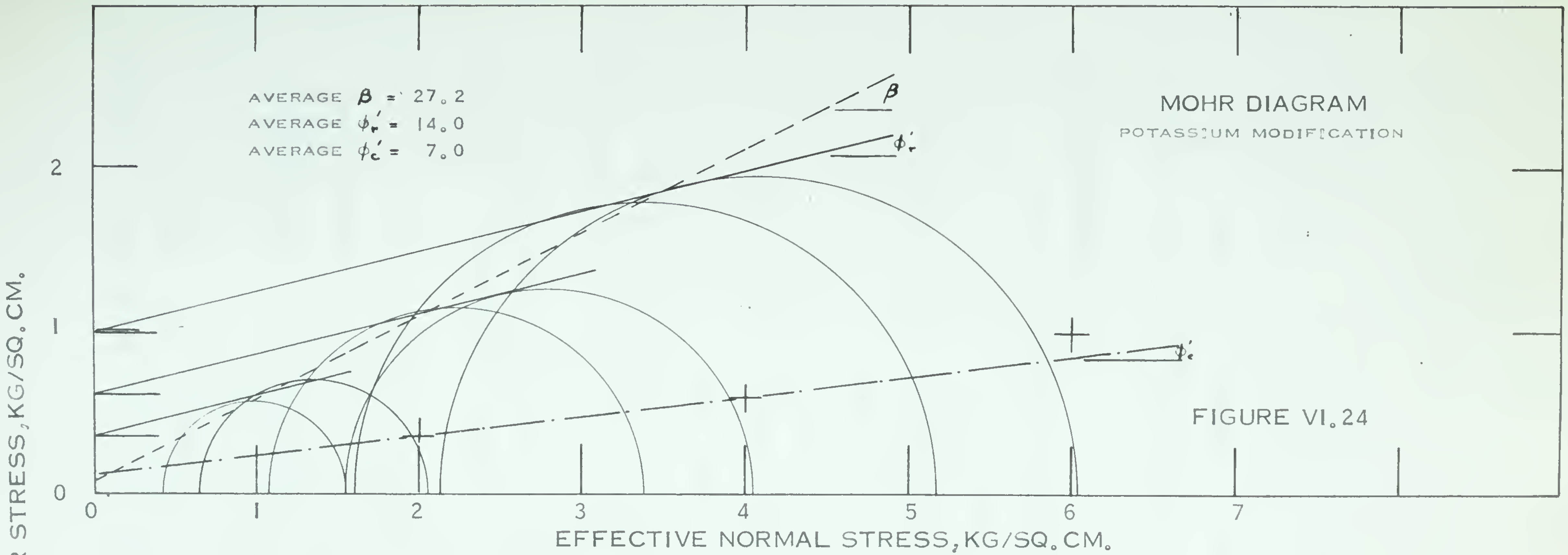


FIGURE VI.24

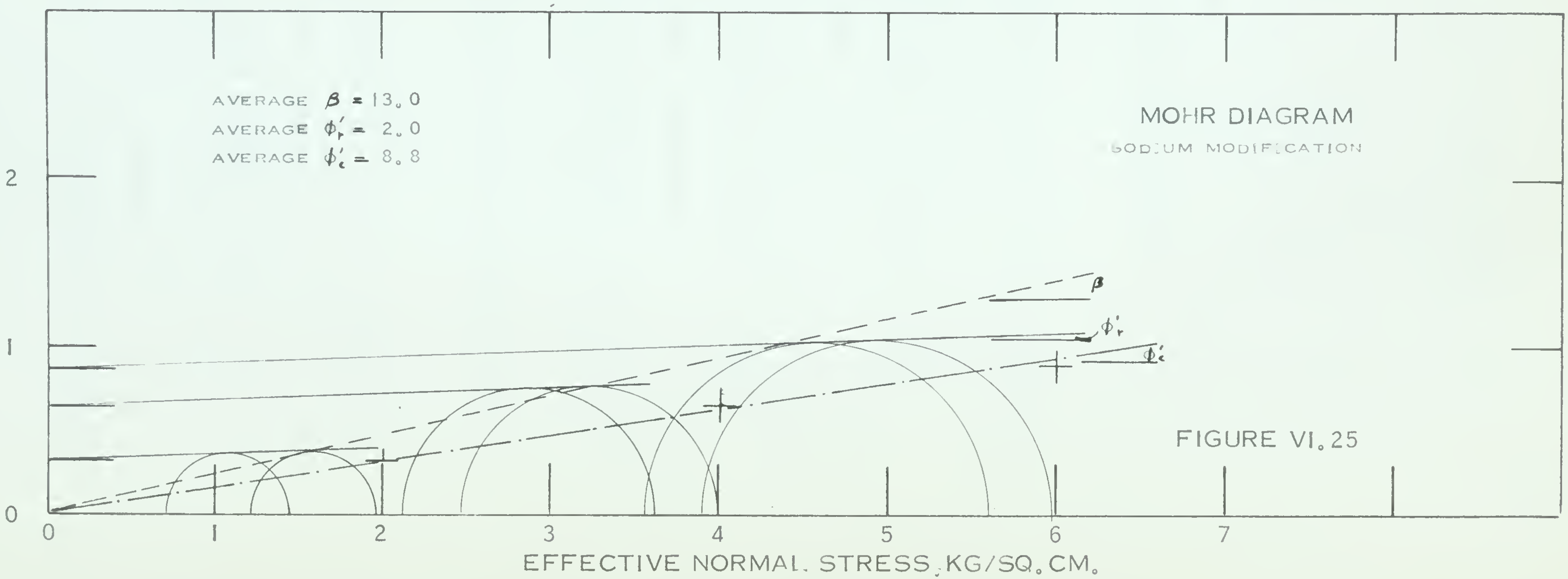


FIGURE VI.25

stress circles from the CFS tests performed for this program. One CFS test yields two failure stress circles; one from the high $\bar{\sigma}_1$ curve and one from the low $\bar{\sigma}_1$ curve. Failure, defined as maximum deviator stress, did not normally occur at the same strain for both curves (see TABLE V.1), thus the soil structures at failure are not the same. Therefore, an analysis based on these circles cannot be expected to yield the same results as a CFS test analysis.

Mohr-Coulomb Analysis

6.24 An effective stress analysis of the results from consolidated-undrained tests on normally consolidated soils based on the Mohr-Coulomb failure criterion theoretically yields a strength envelope whose slope is $\emptyset'$ and whose shear strength intercept is zero. Friction angles ($\emptyset'$) determined by Thomson (1963) and Locker (1963) on modified soils similar to those used for this program are presented in TABLE VI.1, Columns 2 and 3. Zero shear strength intercepts were not generally found although the magnitudes of these intercepts were relatively small. A Mohr-Coulomb analysis of the stress circles obtained from the $\bar{\sigma}_1 \sim \sigma_c$ stress-strain curves of the CFS tests yields the angles designated as β in TABLE VI.1, Column 1.

6.25 It is felt that a comparison between β and $\emptyset'$ is justified by the following reasoning. A study of the data from undrained strength tests performed by Thomson (1963) and Locker (1963) indicates that the ratio of $\bar{\sigma}_1/\sigma_c$ at failure is generally in the order of 1.10 to 1.50, the larger value corresponding to tests performed at low cell pressures. This ratio for the CFS tests of this program was generally

TABLE VI.I

METHODS OF INTERPRETATION OF CFS DATA

AND AVAILABLE COMPARISONS

Soil Type	Mohr-Coulomb Analysis				CFS Test Ave. Max.	Krey-Tiedemann Analysis	
	β	ϕ'	ϕ'	ϕ'	ϕ_ϵ	ϕ'_r	ϕ'_c
	1	2	3	4	5	6	7
Remoulded Edmonton Clay	21.2	21.0	--	24.2	13.4	10.0	8.5
Undisturbed Edmonton Clay	27.4	--	--	--	14.4	11.0	11.0
Magnesium	21.1	23.6	--	--	13.6	12.5	6.5
Calcium	21.0	22.7	23.2	--	12.0	11.1	6.9
Potassium	27.2	22.9	--	--	18.6	14.0	7.0
Sodium	13.0	7.5	12.7	--	2.3	2.0	8.8

1. CFS tests (normally consolidated)
2. From Thomson (1963) (consolidated undrained tests)
3. From Locker (1963) - Low Salts (consolidated undrained tests)
4. Composite Results (APPENDIX D)

6,7. See Figures VI.20 to VI.25

equal to or slightly greater than one. Apart from this difference, conditions obtaining at failure described in terms of moisture content and maximum deviator stress are similar. This reasoning, in effect, assumes a single, linear relationship between moisture content and the logarithm of compressive strength for a given normally consolidated soil tested in accordance with either undrained or CFS test procedures. This assumption is not unreasonable and thus a comparison appears in order.

6.26 The angles obtained from an analysis of the CFS data are in the same order of magnitude as those from undrained tests. The differences are relatively small and the comparison may be considered quite good. Variations shown between the compared angles may be due to small differences in the clay soils used, in the salt contents of the pore water and in the degree of saturation of the exchange complex with a given cation.

6.27 Composite results from consolidated undrained strength tests on Lake Edmonton clay performed for the Graduate Soil Mechanics Laboratory Course at the University of Alberta, Edmonton, are plotted in the form of a modified Mohr diagram in FIGURE D.2, APPENDIX D. The friction angle determined from this plot is given in TABLE VI.1, Column 4. The CFS test result is seen to be slightly lower. The CFS test results plotted on this figure are, however, within the main mass of points and thus both types of test appear to yield similar results in this form.

6.28 FIGURE D.1, APPENDIX D, presents the composite results and the CFS-test results on a moisture content versus log compressive

strength plot and essentially illustrates the same point as contained in the preceding paragraph. That is, a strength analysis of CFS test results in the conventional undrained test manner will yield similar relationships.

Krey-Tiedemann Analysis

6.29 The results of a Krey-Tiedemann analysis performed on the CFS failure stress circles are presented in TABLE VI.1, Columns 6 and 7. The angles of shear strength obtained from this analysis are herein referred to as $\emptyset'_r$ and $\emptyset'_c$. The $\emptyset'_r$ should not be expected to compare exactly with $\emptyset_\epsilon$ obtained from the CFS test (column 5) for the reason outlined in Paragraph 6.18. It is seen that $\emptyset'_r$ is consistently less than $\emptyset_\epsilon$.

6.30 Cohesion intercepts were obtained by extending the average tangent lines (slope tan $\emptyset'_r$) to the shear strength ordinate. The Krey-Tiedemann cohesion line was then found by plotting the cohesion intercepts against the corresponding consolidation pressures. The slope of this line is tan $\emptyset'_c$. This line should pass through the origin as should the Mohr-Coulomb failure envelope. It is seen, however, that both lines have approximately the same intercept for each soil type tested.

6.31 Schmertmann (1960), in a Mohr-Coulomb analysis of his CFS test data, also found that the envelopes did not plot in the conventional manner. The fact that the envelopes did not pass through the origin was attributed to the effects of extrusion preconsolidation, membranes and the net effect of internal and external drains. Machine-extruded

specimens were not used for this program and thus this effect is non-existant. The latter two effects may contribute to the unconventional intercepts although it is difficult to accept these effects as being solely responsible.

6.32 A Krey-Tiedemann analysis cannot yield exactly the same cohesion intercepts and thus cohesion angles as would an analysis of CFS data at strains near maximum deviator stress because of structural differences described earlier. It is of interest, however, to make a simple comparison of cohesion angles from these two methods of analysis. A plot of cohesion at failure from the CFS test data versus consolidation pressure (similar to FIGURE VI.13) yields an average cohesion angle of 8^{o} within a range of 7 to 11^{o}. The average $\emptyset'_c$ from the Krey-Tiedemann analysis on the remoulded soils tested is 7.5 within a range of 6.5 to 8.8^{o}. The analysis on the undisturbed clay was excluded from these figures because only one test was performed. The agreement can be considered quite good. The essential difference between the true CFS analysis and the Krey-Tiedemann analysis is that the former determines cohesion and friction at a given strain with the minimum possible structural difference whereas the latter does not specifically take strain or structure into consideration.

Activity

6.33 A general review of the CFS test data indicates that the sodium modification has the least desirable characteristics while the potassium soil would appear to be the least troublesome for general soils engineering purposes. A calculation of the activity* of the soils of this

* Activity is defined as the numerical value of the ratio of plasticity index to percent sizes less than .002 mm.

program, using the limit values of Hamilton (1961), yields the following values: remoulded clay, 1.05; calcium, 1.06; magnesium, 1.02; sodium, 1.02; and potassium, 0.65. It is of interest to note that the activity value for the sodium modified soil does not indicate the presence of the undesirable characteristics. The low value for potassium does, however, suggest a less plastic behavior relative to the other modifications of this program.

Summary

6.34 This chapter dealt with the results of the CFS tests performed for this program. Cohesion was shown to peak at low strains whereas maximum friction occurred at considerably higher strains for all soils tested except the sodium modifications, where the reverse was found to occur. The mobilized cohesion appeared to be dependent on the consolidation pressure and independent of the adsorbed cations. On the other hand, the angle of internal friction seemed to be related to the type of cation adsorbed and independent of the consolidation pressure.

6.35 Linear relationships between the logarithm of the consolidation pressure, the compressive strength, the peak cohesion and their associated moisture contents were demonstrated. The CFS test was shown to yield two parallel compressive strength lines, one for the condition of normal consolidation and one for slight over consolidation. Structural differences and the resistance of a specimen to volume increase under axial stress were given as possible causes of the occurrence of two separate lines.

6.36 A comparison of the compressive strength lines for the

soils of this program indicated that the type of adsorbed cation affects the position of these lines by shifting them up or down on the compressive strength - moisture content plot.

6.37 A Mohr-Coulomb analysis performed on the CFS test data yielded effective friction angles comparable to those obtained from consolidated undrained strength tests on similarly modified soils. Comparison on a compressive strength-moisture content plot of CFS test data with composite results from previous undrained test investigations on the remoulded Edmonton clay also showed excellent agreement.

6.38 A Krey-Tiedemann analysis was also performed on the CFS test data. The slope of the Krey-Tiedemann cohesion line was shown to be similar in magnitude to the average slope of a line determined from a plot of cohesion at failure from the CFS data versus consolidation pressure.

6.39 The value of activity for the sodium modification did not suggest the presence of the undesirable characteristics displayed by this soil.

CHAPTER VII

DISCUSSION OF RESULTS

Introduction

7.1 It is apparent from the physico-chemical concepts presented in Chapter II that the source and nature of shearing strength of cohesive soils is open to considerable discussion. The concepts presented in this chapter will contain ideas from many postulates and are certainly open to criticism. An attempt will be made, however, to form a trend of thought that will account for the shear strength observations arising from this research program.

Physico-Chemical Effects on Shear Strength

7.2 It is first appropriate to review the physico-chemical variables present in this program. The factors which Lambe (1958) considers to decrease the shear strength of a clay soil were presented in Paragraph 2.19, but are repeated here for convenience in discussion. They are:

1. Cation exchange from high to low valence
2. Exchange from a cation of small hydrated radius to a cation of large hydrated radius
3. Increase in water content
4. Reduction of electrolyte concentration
5. Increase of dielectric constant of pore fluid
6. Increase of pH of pore fluid
7. Decrease of temperature

7.3 The effect of cation exchange from a high to low valence can be evaluated from the results of this program as calcium and mag-

nesium (valence of 2), sodium and potassium (valence of 1) modified soils were prepared. The soil used in preparation of all the modifications is basically the same. Variation in test results can then be partly attributed to changes in the adsorbed cation complex.

7.4 A complicating factor arises, however, as the soils produced were on an average 90 % saturated with a given cation. Thomson, (1963) postulated that the presence of a small quantity of adsorbed magnesium cations may enhance the effects of adsorbed sodium and possibly calcium. The results of the exchange capacity analyses presented in TABLE IV.1 give the impression that the magnesium cation is most difficult to replace as all modifications retained a high proportion of the originally adsorbed magnesium cations. It is not possible to determine the effects, if any, that these magnesium cations may have on the results of this program but it should be recognized as a possible complicating factor.

7.5 Exchange from a cation of small hydrated radius to a cation of large hydrated radius is also a factor that may be considered for this program. The sequence of the cations used in order of increasing hydrated radius is K, Na, Ca and Mg. The cations in the diffuse double layer are in a state of continuous motion (Grim, 1953) and thereby promote disorder in this layer. The size of the hydrated cation partly governs the degree of disorder and thus the extent of the diffuse double layer. It is believed by some authors (Grim and Cuthbert, 1945) that the sodium cation does not hydrate in the diffuse double layer thus allowing a relatively large diffuse double layer to form. This postulate has been accepted in the discussion of sodium diffuse double layers in this

Chapter. Discussion of the nature of the diffuse double layers for the soil-water systems of this program is resumed in Paragraph 7.20.

7.6 The effect of moisture content on compressive strength was evaluated by means of a plot with these coordinates. Moisture content is not a constant during the performance of a CFS test but was shown to be a function of strain and the constant major principal effective stress chosen. As the rate of change of moisture content with strain was shown to be small after attaining maximum deviator stress and all tests after the first two were stopped shortly after attaining maximum deviator stress, the evaluations made were considered in order (Paragraph 5.8).

7.7 The pore water salt concentration within a given series of tests on any one modification was a constant in this program. Hence this is not a significant factor in this program.

7.8 Effects of changes in the dielectric constant of the pore fluid are negligible for the purposes of this program as distilled water was used throughout.

7.9 Determinations of pH were not made during this program. Locker (1963, p.59) indicated that pH likely had a very minor influence on results of tests on the soil of his program. As essentially the same soil was used for this program, pH is not considered to be a factor contributing to strength differences reported herein.

7.10 Recent research by Semchuk (1962) showed that temperature was not a significant factor for the soil used in this program in its

natural remoulded state.

7.11 The main variables that will affect the results of a given CFS test within a series on a specified modification are thus the type of adsorbed cation and the moisture content.

Consolidated Clay-Water Systems

7.12 Paragraphs 2.16 and 2.17 very briefly described the nature of the soil-water system and the various electrical forces present in this system. It was shown that both attractive and repulsive forces exist. Lambe (1953, p.21,22) presents a number of curves of total potential (net effect of attractive plus repulsive forces) versus distance between plates (or particles) that may apply to soil-water systems under various conditions. For the purposes of this discussion, the plot presented in FIGURE VII.1 will be used. The reason for choosing this curve will become apparent by the line of thought followed.

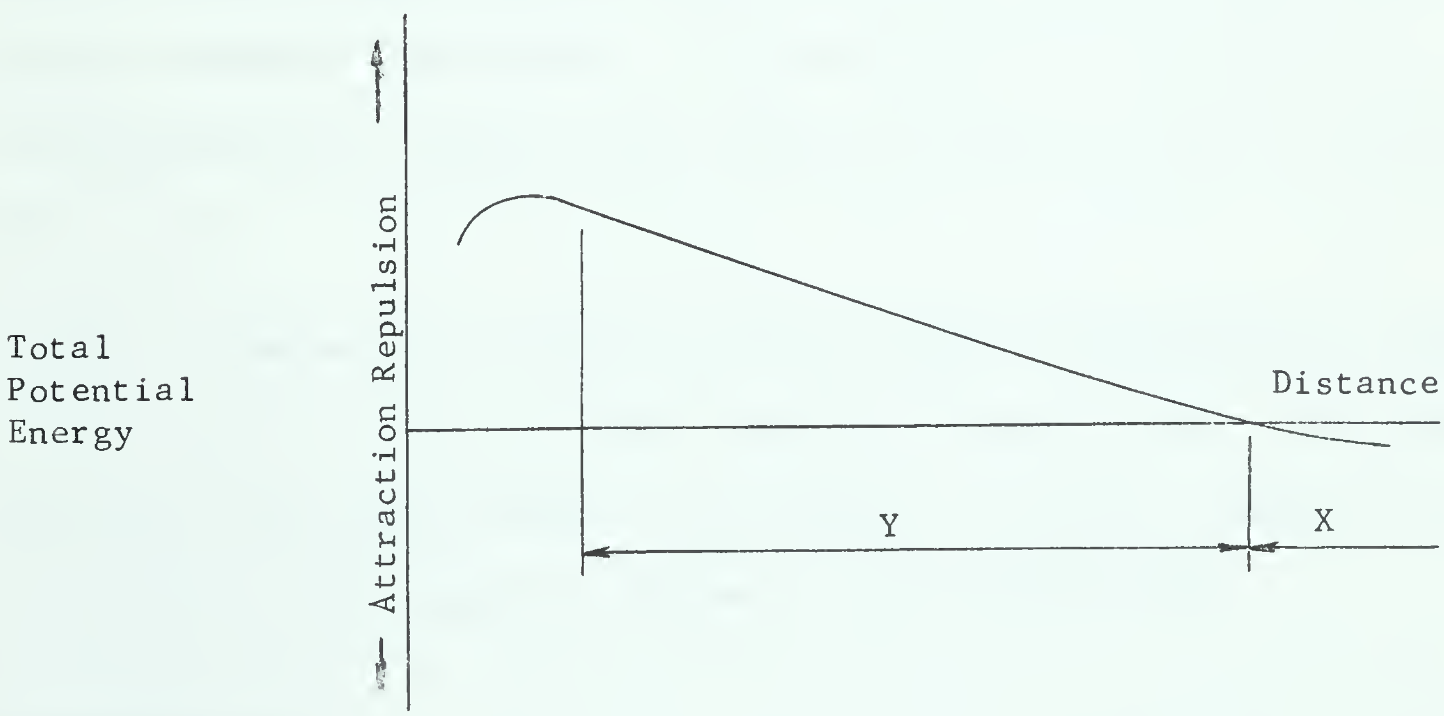


FIGURE VII.1 Interparticle Spacing as a Function of Forces (after Lambe, 1953)

Soil-Water Systems at the Liquid Limit

7.13 Since the liquid limit is the water content at a small but measurable shear strength, Lambe (1953, p.27) indicates that it approximates an upper limit of immobilized water. As the liquid limit is determined under a confining pressure of zero, it appears to be a measure of the net attractive forces present in the soil-water system under the stresses imposed by determination of the liquid limit. The average distance between the clay minerals at this moisture content is relatively large and thus one might consider Michaels' concept (Paragraph 2.13) of pore and adsorbed water to be applicable here. Michaels' felt that the water present in the system allowed interparticle forces to occur over relatively large distances. Lambe (1953) indicates that attractive forces predominate at large distances since repulsive forces decrease with increasing distance at approximately an exponential rate whereas attractive forces decrease as the square of the distance. Thus attractive forces will be larger than the repulsive forces for very large and very small interparticle distances. The preceding reasoning appears to justify the net attractive force shown in FIGURE VII.1 by the distance X.

7.14 The writer's picture of the soil-water system near the liquid limit is one in which some double layer overlapping occurs. The largest portion of the water present in the system is, however, thought to be in a free state (pore water) as opposed to the adsorbed water.

Consolidation

7.15 As consolidation progresses under an external pressure, water

is expelled from the system. More double layers will interact and thus the repulsive forces between the soil particles (clay mineral plus the double layer) increase. Attractive forces also increase but, as shown in the preceding paragraph, do so at a slower rate. When equilibrium conditions are reached, the externally derived effective stress plus the electrical attractive forces (considering a unit area) are equal to the electrostatic repulsion. Average particle spacing associated with consolidation pressures of this program may be considered to be within the distance indicated Y in FIGURE VII.1.

7.16 Structural changes also accompany consolidation under external pressures. The rearrangement of soil particles from a random array toward a more orderly array contributes to compression (Lambe, 1958). Shear strains must occur on a microscopic level due to this rearrangement. Particle spacing and particle orientation thus adjust until electrical equilibrium is established under the external pressure.

7.17 Lambe (1958) indicates that, in the average natural clay that the engineer encounters, essentially all the water is within the double layer. Grim (1958) also states that "in concentrated clay-water systems as in a plastic clay mass, the nature of the adsorbed water is of extreme importance in determining properties because all or almost all of the water is nonliquid". The lowest confining pressure used for this program was 1.6 kg/sq.cm. which may be associated with approximately 32 feet of overburden. Assuming that clays at this depth fall into the category of "the average natural clay" or "a plastic clay mass", it appears that the majority of the water expelled during triaxial

consolidation of the specimens of this program was adsorbed water. From these considerations, the consolidated soils of this program can be pictured as a dense mass of interwoven double layers with possible mineral to mineral contacts.

Characteristics of Soil-Water Systems

7.18 The statement that "the greater the attractive force, the greater the shear strengths the greater the repulsive forces, the lower the shear strength" (Lambe, 1958) is not very clear since the relation between attractive and repulsive forces and the shear strength may be interpreted in two ways. For a given soil system, the greater the external force, the greater the repulsive forces and, since a decrease in void ratio occurs, the shear strength will also be larger. The magnitude of the repulsive forces are dependent, however, on system characteristics, i.e., the nature of the double layer. Thus large double layer may be associated with large repulsive forces, a relatively large void ratio, and thus low strengths. Under a given external pressure, however, attractive forces must also be proportionately large such that the net repulsive force is consistent with the external pressure.

7.19 TABLE II.1 lists various characteristics of cations in the soil-water systems of this program. The discussion in Paragraphs 7.12 to 7.18 purposely omitted inclusion of the effects of these cations as it was intended to describe a system of general applicability. It is now intended to discuss how these cations determine the nature of their respective soil-water systems.

7.20 The preceding paragraphs indicated that a given external

pressure caused an equal and opposing net repulsive force to arise within the soil system. Accepting this, it follows that the same net repulsive force must arise in all clay-water systems if the external pressures are identical. The void ratio at the end of consolidation under a specified confining pressure should then be indicative of the size of the diffuse double layer and also the nature of the soil structure. It is assumed here that the grain size distributions of the systems compared are similar.

7.21 In order to apply the preceding reasoning to the clay-water systems of this program, it is apparent that some measure of the grain size distribution must be taken into account. TABLE VII.1 lists the void ratios at the end of consolidation for the soils of this program. Percent clay sizes from hydrometer analyses performed by Thomson (1963) are also given. These figures are proportional to the number of clay particles and thus are indicative of the number of double layers that may develop. If the void ratio is thought of as a measure of the space available in which the double layers may develop, then the ratio of void ratio to percent clay sizes represents the number of spaces required for the development of a unit double layer.

7.22 As a dispersed clay-water system has a lower void ratio than the same clay in a flocculated system, lower values of void ratio to percent clay sizes may be construed as indicative of a more dispersed system. Considering the computed ratios, one might conclude that the potassium modified soil of this program is more dispersed than the other soils tested. The degree of dispersion of flocculation cannot,

TABLE VII.1

DATA FOR COMPUTATION OF THE RATIO OF VOID RATIO

AFTER CONSOLIDATION TO PERCENT CLAY SIZES

Soil Type	% Clay Sizes[1]	Void Ratio After Consolidation, e_o			$\frac{e_o}{\%\ \text{clay sizes}}$		
		2.00[2]	4.00	6.00	2.00	4.00	6.00
Potassium	57	1.20	1.04	1.02	.210	.183	.179
Sodium	68	1.69	1.46	1.31	.248	.215	.193
Magnesium	48	1.23	1.05	0.95	.256	.219	.198
Remoulded Clay	45	1.23	1.07	1.01	.249	.224	.206
Calcium	46	1.12	1.01	0.93	.268	.233	.220
Undisturbed Clay	44[3]	--	0.91	--	--	.207	--

1. From Thomson (1963)
2. Consolidation pressure, kg/sq.cm.
3. From TABLE III.1, page 26.

however, be determined.

7.23 It is not surprising that a potassium modified soil should demonstrate a relatively dense system. The size of the potassium cations are such that they just fit into crystal lattice structure formed by the bases of the tetrahedrons. This selective process is known as "potassium fixation". The result of this fixation process is a partial satisfaction of the clay mineral negative surface charge with a disproportionately small contribution to the thickness of the double layer. Some hydrated potassium cations are, however, adsorbed but because the potassium cations have a relatively small hydrated radius (TABLE II.1) their contribution of the thickness of the double layer is also relatively small. Thus the tight interparticle bonds proposed by Grim (1953, 1958) are not difficult to imagine.

7.24 It was indicated earlier (Paragraph 6.20) that an increase in salt content of the pore water of the sodium modification would result in a considerable decrease in moisture content, or void ratio, at a given consolidation pressure. Locker's work (1963, p.72) indicates that, at a salt content comparable to those of this program (7 me/100 gms. a.d.s.), a void ratio of approximately 1.30 could be expected. Using this figure instead of 1.69, a value of 1.91 is obtained for the value of void ratio to percent clay sizes. Comparing the four modifications at similar salt contents now indicates that the sodium modification is the more dispersed system, calcium and magnesium are more flocculated systems and potassium is inbetween these. This order is compatible with that shown in TABLE II.1.

7.25 The observations drawn from consideration of the ratios given in TABLE VII.1 might be considered purely analytic as the results of hydrometer analyses vary with method of pre-treatment and dispersing agent used. The percent clay sizes for the first five soil types listed were, however, all determined by Thomson (1963) the pre-treatment and dispersing agent used being the same for all determinations. For these reasons, the percent clay sizes shown should be representative of the relative amounts in the soils listed thus the observations of the preceding paragraphs do not appear to be invalidated.

7.26 The analysis made in Paragraph 7.22 indicated that the calcium, magnesium, and sodium modifications and the natural clay may have soil-water systems tending toward the flocculated condition. Adsorbed calcium and magnesium cations in a soil-water system are associated with thin double layers and a sharp transition to non-oriented water whereas adsorbed sodium cations are associated with relatively thick double layers and a gradual transition to free water. It would appear that the former cations may generate flocculated soil-water systems but, as sodium systems are generally considered to be of a dispersed nature, the postulates presented based on the value of void ratio to percent clay sizes appear inconsistent in the case of the sodium modification.

7.27 It is felt, however, that the reasoning based on the calculated ratios must be applied to either the sharp or the gradual transitions of the adsorbed water but not both. This would explain why it appeared to apply when all modifications were compared at the same pore water

salt content (Paragraph 7.24). At this salt content (approximately 7 me/100 gms. a.d.s.) the sodium oriented layer is probably suppressed in such a way that the transition to non-oriented water is relatively abrupt. Thus the apparent anomaly is explained and the proposal of a dispersed sodium soil-water system at the salt content of this program is not unreasonable. If this is accepted, then a comparison of the ratio of the void ratio to percent clay sizes for the sodium soil of this program to that at the higher salt content indicates that the former must have a larger diffuse double layer. The association of a large double layer at very low salt contents for the sodium modified soils is consistent with the findings of Thomson (1963) and Locker (1963).

Considerations of Cohesion

7.28 Cohesion has been variously considered as an intrinsic pressure or the result of plastic deformations (Chapter II). In this chapter, soil-water systems have been described mainly in terms of net electrical forces and thus this concept will be maintained in a consideration of cohesion.

7.29 When a soil specimen is consolidated in the triaxial cell some clay mineral reorientation must take place. On a microscopic scale this must be accompanied by shear stresses and strains. This can be thought of as a reorganization of the electrical attractions and repulsions. After some time, an equilibrium is established at a reduced void ratio which is compatible with the external confining pressure.

7.30 It was previously postulated that the external confining pressure and the internal net repulsive forces were equal. As the peak cohesion measured during a CFS test was found to be a function of the consolidation pressure, it appears that this peak cohesion is a measure of the net repulsive forces that arise due to the consolidation pressure.

7.31 If this is the case, the cohesion should be independent of system characteristics such as the type of adsorbed cation or the concentration of the salts in the pore water. For the soils of this program, this appears to be true. Cohesion was shown to be directly related to the confining pressure and appeared to be relatively unaffected by system characteristics.

7.32 The peak cohesion measured in the CFS test may be thought of as the inertia of the established soil-water system to imposed shear strains. As the specimen prior to the CFS test has come to equilibrium under a given consolidation pressure a certain unit force or pressure must be required to overcome this stable condition. Thus the peak cohesion is considered to reflect this required pressure.

7.33 Once the inertia of the system has been overcome, cohesional resistance decreases slightly and then continues to decrease slowly as a function of strain. The cohesional resistance at high strains (in excess of 8 to 10%) still appeared to be related to the consolidation pressure (FIGURE VI.13). Because this resistance continues to decrease slowly with strain, there appears to be a cohesion-to-friction transfer as measured which is primarily a function of strain. As

cohesion is pictured as an electrostatic manifestation and friction is also considered to be partly so (described in the following paragraphs), this transfer may simply be a mechanical separation arising from the test technique.

Considerations of Internal Friction

7.34 Rosenqvist (1955) has presented a particularly clear picture of the causes of friction within a clay soil (Paragraph 2.12). Friction was considered to be due to macro- and micro- dilatency to which was added a non-dilatant friction component which accounted for electrical forces arising due to the normal stress. It is felt that a similar picture is applicable to the frictional component of shear strength measured in the program of this thesis.

7.35 Macro- and micro- dilatency might be pictured simply as particle interference. In the soils of this program, 30 to 55 percent of the soil particles present are of silt and sand sizes and thus one would expect some interference to arise from interaction of these particles during shear strain. It is difficult to visualize whether or not the clay particles interfere in a dilatant manner. It is felt, however, that the clay particles will interfere with one another as a result of particle reorientation which accompanies shear strain.

7.36 The particle reorientation which accompanies shear strain must be associated with interparticle forces whose relationship with one another is being changed. This must require an expenditure of work or energy, the "amount" of this work varying with the nature of the soil-water system. Total mobilized frictional resistance at any strain

may thus be considered the sum of the energies used to overcome particle interference and reorientation at that strain.

7.37 A gross measure of the nature of the soil-water system is the void ratio and thus on a macroscopic scale the work expended to mobilize frictional resistance should vary with the void ratio. As the effective normal stress increases as the void ratio decreases, the mobilized frictional resistance should be proportional to the effective normal stress. The generally accepted relationship that forms the basis for frictional resistance computations of this program comes from the Coulomb-Hvorslev failure equation. This states that the frictional resistance is equal to the effective normal stress times the tangent of an "angle of internal friction". Hence the increased work required to mobilize frictional resistance at a decreased void ratio manifests itself as an "angle of internal friction" on the Mohr diagram.

7.38 Schmertmann's reasoning which justified the use of the Mohr diagram at any strain was presented in Paragraph 6.21. The angle of internal friction at any strain was defined as "the angle whose tangent is the ratio of the change in shear stress to the change in normal intergranular stress occuring on the plane of Mohr envelope tangency at that strain, during a stress change occurring without significant change in soil structure". The various slopes of the angle of internal friction versus strain plots (FIGURES VI.7, 9, 11) reflect a rate of change of energy input (or demand). That is, as the effective normal stress is increasing during axial compression, the effective shear stress is, at first, increasing at a faster rate($\emptyset_{\epsilon}$ is increasing).

As failure is approached, a point is reached such that the ratio of the change in effective shear stress to the change in effective normal stress becomes a constant. This ratio is the tangent of the maximum angle of internal friction.

7.39 On a microscopic scale, the mobilization of frictional resistance was interpreted as due to both interparticle interference and a reorientation of soil particles and thus electrostatic forces. The individual contribution of either one of these factors to the total mobilized frictional resistance is difficult, if not impossible, to assess.

7.40 Based on the CFS tests of this program, one might, however, be able to postulate which of the two contributing factors appears more important. As cohesion was found to be similar in all soils tested at a given confining pressure, a comparison of the maximum angles of internal friction in conjunction with pertinent system characteristic may be of value.

7.41 Adsorbed potassium was described as resulting in relatively thin double layers and thus, under a given confining pressure, a relatively dense (low void ratio) soil specimen. It would be suspected that interparticle interference is a major portion of the frictional resistance mobilized. On the other hand, adsorbed sodium was associated with a relatively large double layer and thus a high void ratio under a given confining pressure. The frictional resistance mobilized may in this case reflect the work necessary to move electrostatic forces with interparticle interference contributing little to the frictional

resistance.

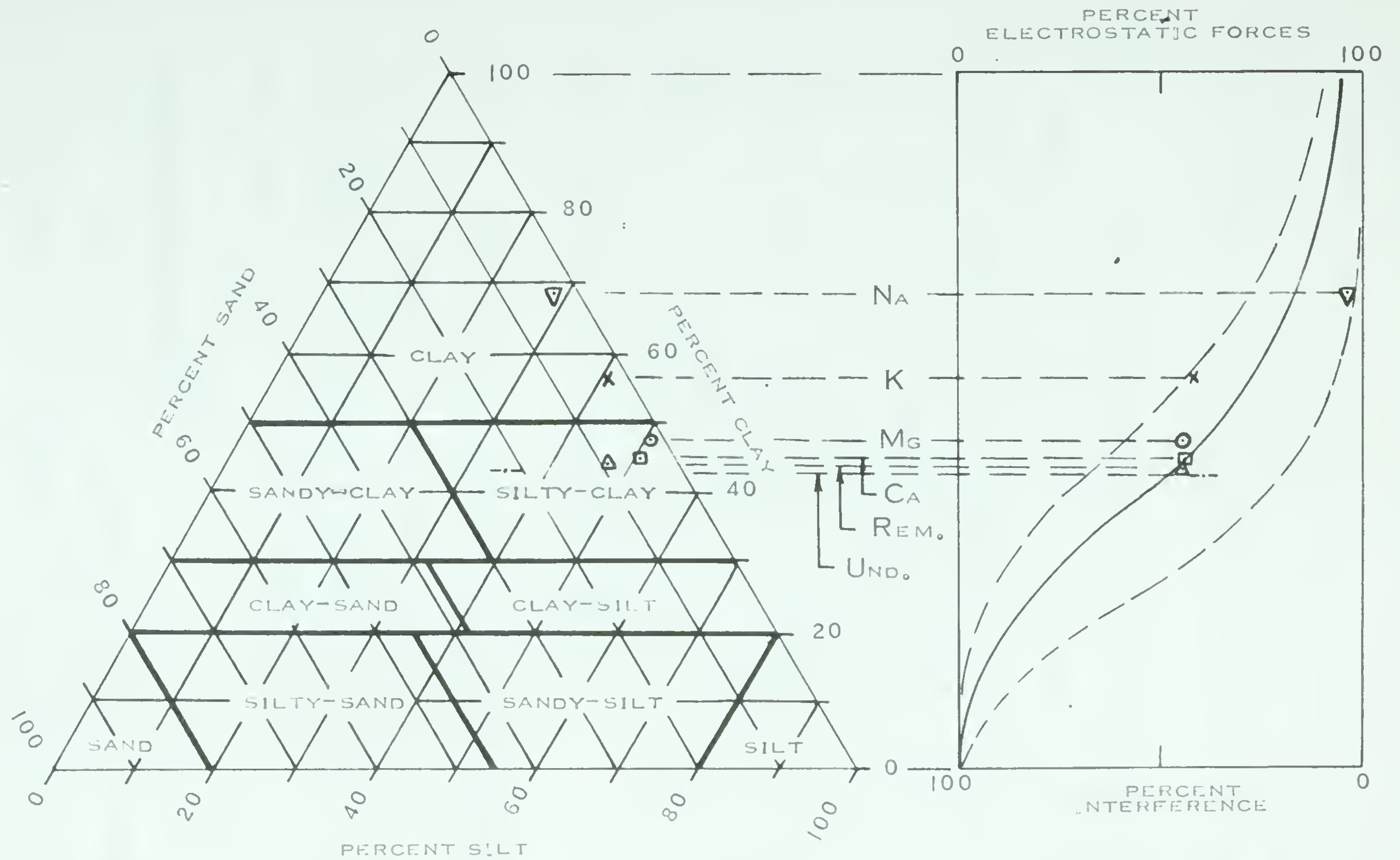
7.42 The nature of the transition from oriented to free water may also be of some importance. The well defined transition boundary displayed by adsorbed potassium cations may be much more difficult to disrupt by shear strain than the more gradual transitional boundary of the sodium soil.

7.43 Thus the high angle of internal friction of the potassium soils tested may be a manifestation of large contributions from both interparticle interference and electrostatic bonds that resist alteration. On the other hand the low angle of internal friction of the sodium soils may be primarily the result of movement and breaking of weak electrostatic bonds.

7.44 The double layers of the calcium, magnesium and natural soils tested are probably slightly larger than the potassium soils as the fixation process is not a part of calcium and magnesium bonding. Thus the resistance provided by electrostatic forces may not be as strong as that in the potassium soils.

7.45 The concept of frictional resistance being a combination of interparticle interference and reorientation of electrostatic forces appears to allow a continuity of strength theory through "pure" sands to "pure" cohesive soils. This concept is illustrated in FIGURE VII.2.

7.46 For a given soil, a point is obtained on the triangular chart that determines the classification of this soil. From this point, a



NOTE - TRIANGULAR CLASSIFICATION CHART FROM LOWER MISS. VALLEY DIVISION, U.S. CORPS OF ENGINEERS. BASED ON SIZE LIMITS :

SAND... 2.0 TO 0.05 MM
SILT... 0.05 TO 0.005 MM
CLAY... LESS THAN 0.005 MM

SIZE LIMITS FOR SOILS OF THIS PROGRAM ARE ACCORDING THE M.I.T. GRAIN SIZE SCALE :

SAND... 2.0 TO 0.06 MM
SILT... 0.06 TO 0.002 MM
CLAY... LESS THAN 0.002 MM

FIGURE VII.2 SCHEMATIC REPRESENTATION OF CONTRIBUTING FACTORS TO FRICTIONAL RESISTANCE IN SOILS

line is extended horizontally to the rectangular chart. The intersection of this line and the band shown indicates the approximate contributions of electrostatic forces and dilatancy to the frictional resistance of this soil to shear. The curve shows that at very low clay contents, the dilatancy component is predominant and at large clay contents, the electrostatic forces are predominant.

7.47 The band is drawn to account for the extremely variable nature of different soils and also to acknowledge the effects of different adsorbed cations and salt contents on a given type of soil. The former effect may be illustrated by considering soils with similar clay contents but different sand and silt size contents. An increase in the silt size content would tend to increase the electrostatic force contribution to frictional resistance. The latter effect is illustrated by the writer's opinion of the position of the soil modifications of this program on the rectangular chart.

7.48 The curves presented are rough approximations, however, the concept appears reasonable and does provide a logical picture of the source of frictional resistance in soils. The actual shape of the curves must be left to future research.

Cohesion and Friction

7.49 The mobilized cohesion appeared to reflect an electrical balance which was considered to be a function of the consolidation pressure and independent of system characteristics. Mobilized friction, on the other hand, was considered to be a manifestation of the interparticle interference and reorganization of electrostatic forces.

As both cohesion and friction appear to be a function of electrostatic forces, it is extremely difficult to separate these two components in a clear manner.

7.50 Since the mathematical determination of cohesion in the CFS test procedure is a function of the angle of internal friction, these two components cannot be considered unrelated from this point of view.

7.51 The conclusion may be justifiably drawn here that these two components are interrelated in such a way that the definition of two separate components is unrealistic from a practical point of view. Following this line of thought, the strength of a given type of soil would be simply defined by a plot of the logarithm of compressive strength versus moisture content. This is in accord with the proposal made by Schmid (Paragraph 2.14) based on energy concepts.

7.52 This conclusion does not, however, detract from the value to be derived from research based on shear strength component separation. The CFS test appears to yield mechanically independent components and is thus an ideal research technique for this purpose. It is felt that research of this nature yields a much clearer picture of the mechanisms contributing to the shearing strength of cohesive soils. Furthermore, it allows conclusions, such as the preceding one, to be drawn.

Summary

7.53 This chapter was concerned with a physico-chemical interpretation of the results of the CFS tests presented in Chapter VI. A

review of the factors that may affect the CFS test results of this program indicated that the main variables present were the type of adsorbed cation and the moisture content.

7.54 A soil-water system at the liquid limit was pictured as one in which some double layer overlapping occurred. The net electrical force was considered to be attractive in nature. The same system under confining pressures similar to those used in this program was visualized as a dense mass of interwoven double layer with possible mineral to mineral contacts. The consolidation process was thought of as a reorganization of electrical attractions and repulsions. It was postulated that a given external pressure caused an equal and opposite net electrical repulsive force to arise within all clay-water systems.

7.55 The numerical value of the ratio of void ratio after consolidation to the percent clay sizes was considered to represent the number of spaces required for the development of a unit double layer. Lower values of this ratio were thought to be indicative of a relatively dispersed system and larger values to be indicative of a relatively flocculated system. Taking into consideration the type of adsorbed cation and the nature of its diffuse double layer, a comparison of the computed ratios indicated that the sodium modification was the more dispersed system, calcium and magnesium soils more flocculated systems with potassium soil being inbetween these two types of systems.

7.56 The peak cohesion measured in the CFS test was considered to be a measure of the net repulsive forces that arise due to the consolidation pressure and was considered to reflect the inertia of the system to imposed shear strain. Mobilized friction was felt to be a manifestation of the interparticle interference and the re-organization of electrostatic forces.

7.57 The concept of frictional resistance being a combination of interparticle interference and reorganization of electrostatic forces appeared to allow a continuity of strength concepts through "pure" sands to "pure" cohesive soils.

7.58 It was concluded that the cohesion and friction components were so closely related that the definition of two separate components was unrealistic from a practical point of view. However, research based on shear strength component separation that may yield a clearer understanding of the mechanisms of shearing strength in cohesive soils is of major importance. The CFS test appears to be an ideal technique for this purpose.

CHAPTER VIII

CONCLUSIONS

8.1 This thesis dealt with a series of CFS tests on remoulded, normally consolidated specimens of a highly plastic, lacustrine clay. The samples tested comprised the natural soil and sodium, potassium, magnesium and calcium modifications. From the test results and conclusions presented in Chapters VI and VII, it appears that the conclusions presented in the following paragraphs are justified.

8.2 For the soils of this program, cohesion as defined by the CFS test was shown to be a function of consolidation pressure and appeared to be unaffected by system characteristics. Cohesion was also shown to peak at relatively low axial compressive strains. It is postulated that the peak cohesion is a measure of the net repulsive forces that arise due to the external confining pressure and that it may be considered to reflect the inertia of the system to imposed shear strains.

8.3 Mobilized frictional resistance as defined by the CFS test was shown to reach its maximum value at relatively large axial compressive strains with the exception of the sodium modifications. For the soils of this program it appeared to be related to the type of cation adsorbed and independent of the consolidation pressure. It is postulated that the mobilized frictional resistance of a given soil is a manifestation of interparticle interference and the reorganization of

electrostatic forces.

8.4 The postulation of this thesis concerning the nature of the friction component appears to allow a continuity of strength concepts through "pure" sands to "pure" cohesive soils. It is suggested that shear resistance grades from friction and dilatency in sands to interparticle forces in clays. Soils comprising mixtures of sands and clays derive their shear resistance from a combination of interference and interparticle forces, the major contribution being attributable to the soil type predominating.

8.5 The average maximum angles of internal friction for the soil modifications tested are presented in TABLE VI.1. The high angle of internal friction of the potassium modification was considered to be a manifestation of large contributions from both interparticle interference and electrostatic forces that resist alteration. The lower angles of internal friction of the calcium, magnesium and natural soils were thought to be due to a smaller contribution from the electrostatic forces. The extremely low angle for the sodium soil was felt to be primarily the result of movement and breaking of weak electrostatic forces. It is suggested that the nature of the frictional resistance of a soil is primarily determined by the type of diffuse double layers present and that the diffuse double layers are thin for high angles of internal friction and thick for low angles.

8.6 It is concluded that the defined cohesion and friction components are so closely interrelated that the definition of two separate

components is unrealistic from a practical point of view. It is thus suggested that simplest and most practical method of soil strength presentation is in the form of a plot of moisture content versus the logarithm of compressive strength. However, an invaluable insight into the shear strength of cohesive soils can be gained by research utilizing the two component approach. The CFS test appears to yield mechanically independent strength components and is therefore an excellent research technique.

8.7 The CFS test was shown to yield two parallel compressive strength lines; one for the condition of normal consolidation and one for slight over consolidation. It is proposed that this separation is due to structural differences in the two states of consolidation and also due to the resistance of the soil specimen to volume increase under stress.

8.8 The numerical value of the ratio of void ratio after consolidation to percent clay sizes was considered to represent the number of spaces required for the development of a unit double layer. A comparison of the computed ratios taking into consideration the type of adsorbed cation and the nature of its diffuse double layer lead to the postulate that the sodium modified soil of this program is the more dispersed system, the calcium and magnesium soils more flocculated systems with the potassium soil being a combination of these two types of systems.

8.9 Low pore pressure reactions without back pressure are considered to be primarily caused by expansion of lines and the pore pressure measuring system. Additional consolidation initiated by the

increase in cell pressure is considered to contribute to the magnitude of the pore pressure reactions of the more permeable soils tested.

8.10 Under a single cell pressure increment, burette measurements of the volume change of the triaxial specimens are considered to be too large due to expulsion of entrapped air and extraneous water.

CHAPTER IX

RECOMMENDATIONS

9.1 Arising from the test procedures and results of this thesis the following recommendations are presented for future work.

9.2 It is recommended that research be performed on over consolidated soils. It is suggested that this program take the form of a number of CFS tests on both normally and over consolidated specimens of a given soil at various confining pressures and over consolidation ratios.

9.3 It was indicated in Paragraph 6.10 that the peak cohesion measured in the CFS test may be a function of the plasticity characteristics of the soil. It was postulated that the mobilized frictional resistance of a given soil is a manifestation of interparticle interference and the reorganization of electrostatic forces. The contribution of each of these factors to frictional resistance of a given soil was considered to be a function of a number of variables. Definition of these variables would, in turn, determine the plasticity characteristics of the soil in question. It is recommended that effect of plasticity characteristics on cohesion and friction be investigated by a series of CFS tests on soils having predetermined quantities of clay and sand or clay and silt. Interesting trends in specific gravity, the Atterberg limits and consolidation characteristics would also be revealed.

9.4 Combinations of clay minerals other than those of this program (TABLE III.1) should also be investigated. This program may consist of CFS tests on soils having similar plasticity characteristics with varying types and proportions of the clay minerals and adsorbed cations or could, as indicated in the preceding paragraph, also investigate the effect of varying plasticity.

9.5 Pilot model tests should be performed prior to future programs utilizing the CFS test as technique is extremely important.

9.6 All CFS tests should be performed with a volume change indicator in the pore pressure line to obviate problems involving volume change measurements during the CFS test.

9.7 Volume changes during triaxial consolidation as measured by a burette were shown to be in error (Paragraph 5.4). Either or both of the following corrective measures are suggested:

(a) An initial consolidation under a very small load, for example, 0.05 kg/sq.cm., may expell any excess air or water in the system.

(b) A record of the quantity of air expelled during triaxial consolidation should be made and applied as a correction.

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APPENDIX A

BACK PRESSURE, PORE PRESSURE REACTION

AND DETAILS OF THE CFS TRIAXIAL TEST

APPENDIX A

BACK PRESSURE, PORE PRESSURE REACTION AND DETAILS OF THE CFS TRIAXIAL TEST

A.1 In the following paragraphs, specific references are made to pieces of apparatus; for example, valve 2 or screw control C. The reader is directed to FIGURE A.1, Schematic Diagram of Triaxial Apparatus During CFS Test,for clarification of these references. Two methods of test were used, herein referred to as Method 1 and Method 2. The essential difference between these two methods was the position of the volume change indicator in the layout of the triaxial apparatus. For Method 1, the volume change indicator was placed in the cell pressure line while for Method 2 it was placed in the pore pressure line. Both positions are indicated in FIGURE A.1. Method 1 was used for the first 12 tests performed and Method 2 for the remaining 5 tests.

<u>Back Pressure - Method 1.</u>

A. 2 After consolidation had taken place, the triaxial cell was placed in the loading press and the load cell set in position. Up to this point, pressure was maintained in the triaxial cell by means of constant pressure cell II. The same pressure was put into the volume change indicator using constant pressure cell I. The volume change indicator could now be zeroed or set at any desired value simply by

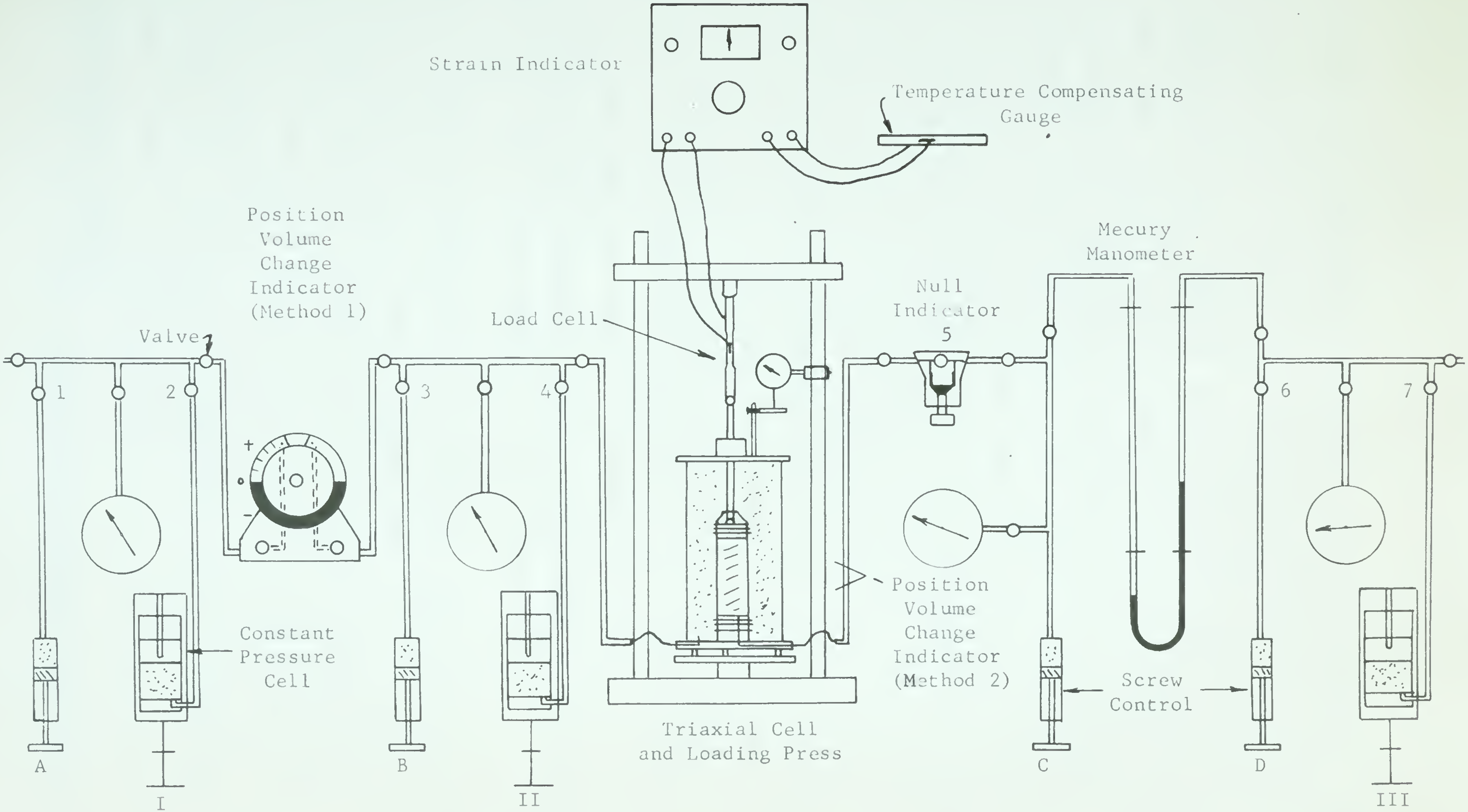


FIGURE A.1 SCHEMATIC DIAGRAM OF TRIAXIAL APPARATUS DURING CFS TEST

closing valve 2 and making the adjustments using screw control A. Movement of water as a result of this adjustment was taken up by constant pressure cell II. At this point the volume change indicator was set at a high positive value. Valves 2, 3 and 4 were then closed. Valve 7 was closed and 2 kg. per sq. cm. was maintained in pressure cell III until it was required. The pressure on the mercury column at this point is atmospheric. Using screw control A, the cell pressure was increased by 2 kg. per sq. cm. while the pore pressure was kept nulled (valve 5 is closed) using screw control C. The pore pressure and back pressure continued to build up at a decreasing rate with time. This procedure is essentially a pore pressure reaction test with no back pressure. After 10 to 15 minutes, during which time the pore pressure generally attained 0.75 to 1.5 kg. per sq. cm., valves 5 and 7 were opened. As the specimen appeared to indicate an affinity for water, the pore pressure was set slightly above 2 kg. per sq. cm.. The specimen was left in this manner over night and equilibrium was reached by the following morning. Using this method, no measurement of the quantity of water taken in by the specimen, if any, could be made.

A.3 When the cell pressure was increased by 2 kg. per sq. cm., the volume change indicator showed a negative quantity. This would represent a volume increase of the cell and lines, a volume decrease of the sample, dissolving of air, a leak on the triaxial cell side of the volume change indicator or the sum of any or all of these. The immediate total change was in the order of 9 to 16 c.c. and volume change continued to occur very slowly.

Back Pressure - Method 2.

A.4 Using this method, the need for screw control A and constant

pressure cell I was eliminated. The volume change indicator was placed in the pore pressure line as shown in FIGURE A.1. The cell pressure was now increased using screw control B and maintained by constant pressure cell II while the pore pressure build-up with time was measured as in Method 1. As before, after 10 to 15 minutes, the pore pressure was set slightly above 2 kg. per sq. cm. and the sample allowed to reach equilibrium conditions over night. It was found, however, that the specimen intake of water because of back pressuring was only in the order of 0.05 c.c. as measured by the volume change indicator. Thus the volume change measured in Method 1 was probably due to the factors mentioned, excluding a volume decrease of the specimen. A leak in the system would account for the continued slow volume change measured and would also account for the need to set the pore pressure above 2 kg. per sq. cm. The latter leak was found to be through screw control C.

Pore Pressure Reaction

A.5 Prior to this test, valves 5 and 2 (Method 1) or 4 (Method 2) were closed and an additional 1 kg. per sq. cm. placed on constant pressure cell I (Method 1) or II (Method 2). The cell pressure was quickly increased by 1 kg. per sq. cm. using the appropriate screw control, the constant pressure cell brought in, and the increase in pore pressure with time measured on the mercury manometer using screw control C to keep the null indicator balanced. Readings were taken at time intervals of 0.1, 0.5, 1, 2, 3, 4 and 5 minutes. At this time, pore pressure had generally settled down and indicated an 80 to 100 per cent reaction. The procedure was simply reversed and readings taken at the same time intervals when the cell pressure was dropped to its

original value. Data sheets illustrating the pore pressure reaction test are included in APPENDIX C.

The CFS Test

A.6 The following description of the CFS test method applies directly to the tests on the more permeable specimens tested; that is, the calcium, magnesium and potassium modifications and the natural soil. A separate paragraph will deal with the CFS tests performed on the sodium modifications.

A.7 The test was started as a normal undrained test with pore pressure measurement to approximately 0.05% strain. This was done to insure that the piston was seated and to adjust the strain dial accordingly.

A.8 Points 1 and 2 (FIGURE A.2) were obtained on the high $\bar{\sigma}_1$ curve (approximately 100% σ_c). This was done by increasing the pore pressure

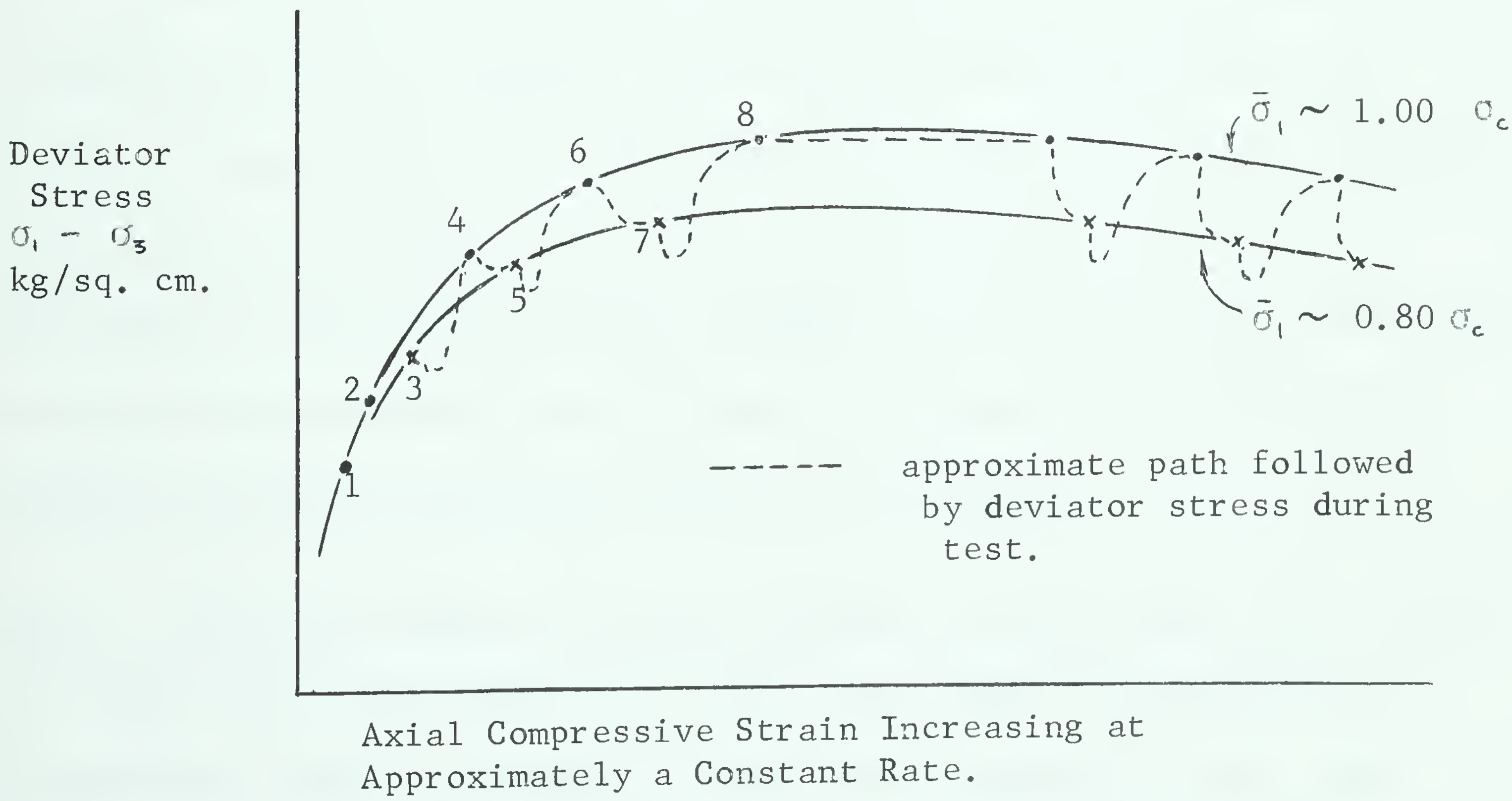


Figure A.2 Deviator Stress versus Axial Compressive Strain for the CFS Test.

at the base of the specimen using screw control C with null indicator valve 5 open. In this portion of the deviator stress curve, the pore pressure was increased to approximately 1.5 times the expected required value, held there for about 12 minutes and then nulled and measured over a period of 8 to 10 minutes. This interval was usually found to be sufficient for the calcium, magnesium and natural specimens while the potassium samples generally required a little more time for adjustment and equilization.

A.9 Point 3 was then obtained on the low $\bar{\sigma}_1$ curve (approximately 80% σ_c) by increasing the pore pressure by about 1.5 times the difference between the $\bar{\sigma}_1$ values selected for each curve. This pressure was maintained for a period of time ranging from 3/4 to 1 1/2 hours. Periodic checks were made during this time to check the pore pressure and deviator stress attained. When it appeared that the specimen was on the lower $\bar{\sigma}_1$ curve, the pore pressure was nulled and measured over an interval of 10 to 15 minutes. If the point determined fell on the curve, a further 8 minutes (.05%) was used to check the point, and this last point used to define the curve. If the point was not on the desired $\bar{\sigma}_1$ curve, the appropriate changes in pore pressure were made and sufficient time allowed for the sample to accomodate the change before another point was determined using the null method.

A.10 To obtain point 4, the pore pressure was decreased by the amount 1.5 $\Delta\bar{\sigma}_1$ using screw control C. From 1 to 2 hours was generally required for a "hop" to be made to the higher constant $\bar{\sigma}_1$ curve. The dashed line in FIGURE A.1 indicates the path followed by the deviator

stress during "hops". When the pore pressure is decreased, the deviator stress immediately falls and then requires a length of time dependent primarily on the rate of strain, and the magnitude and rate of volume change to build up again to a value consistent with the new pore pressure and desired $\bar{\sigma}_1$. The method of control and measurement of pore pressure and deviator stress was similar to that described in paragraph A.9.

A.11 This method of "curve-hopping" was continued to approximately 5.5 percent axial strain. As the tests were started at about 0830 hours, this percent strain was attained by approximately 2300 hours under the rate of strain employed (1% per 155 minutes). The test was left running through the night and "curve-hopping" recommenced at about 0630 hours the following morning. Approximately 3 percent axial strain was missed during the 7 1/2 hour period over night. It was found that the pore pressure fell during the night if the manometer was left at the final pore pressure value measured. This fall was most likely due to leaks in the system as described previously. With the apparatus used, it was found necessary to increase the pore pressure by approximately 15% of the final reading in order to fall on the same $\bar{\sigma}_1$ curve in the following morning.

A.12 The "hopping" procedure described in paragraph A.9 was continued until both curves had passed maximum deviator stress and were adequately defined.

A.13 Throughout the test pore pressure, deviator stress, volume

change and time readings were taken and calculations for $\bar{\sigma}_1$, $\bar{\sigma}_3$ and $\bar{\sigma}_1/\bar{\sigma}_3$ made immediately. This was necessary in order to determine exactly where the specimen was with regard to the desired constant $\bar{\sigma}_1$. Plots of pore pressure, deviator stress, volume change, minor principal effective stress and stress ratio versus strain were kept continuously to insure no gross errors occurred.

A.14 After approximately 2 percent axial strain, the volume change versus percent strain curves became quite regular and predictable. By knowing quite closely what volume change reading to expect when the specimen is on the desired constant $\bar{\sigma}_1$ curve, the technician could use this as a guide as to when to null and measure the pore pressure.

<u>CFS Test On Sodium Modifications</u>

A.15 Tests on the relatively impermeable sodium modifications were started in the same manner as the tests previously described. The rate of strain, however, was 1 percent axial strain per 1393 minutes, approximately 9 times as slow. This rate of strain and the rate of volume change of the sodium specimens generally allowed 3 points to be determined on the deviator stress versus strain curve on the first day and one to three points on each succeeding day. As failure defined as maximum deviator stress normally occurred in the order of 4 to 5% axial strain, the test lasted 5 to 6 days. Procedures outlined in preceding paragraphs are generally applicable if the times indicated are adjusted accordingly. Measurement of pore pressure alone which, for the relatively permeable specimens, required 15 to 25 minutes, now required 1 1/2 to 3 hours. As would be expected, volume changes also

took considerably longer to occur.

A.16 At the end of the test, the loading press was shut off and valve 7 and valve 2 (Method 1) or valve 4 (Method 2) were closed, valve 5 remaining shut. Cell pressure, by means of screw control I (Method 1) or II (Method 2) was backed down simultaneously with pore pressure using screw control C. As valve 7 was closed, back pressure fell with the pore pressure. Keeping the pore pressure nulled to prevent any volume change, pore pressure was brought down to atmospheric pressure. The pore pressure line was disassembled and the lead to the sample immediately plugged. The remainder of the cell pressure was taken off the sample and the entire apparatus dismantled. Factors noted at the end of test were: angle of shear plane, if visible, wet weight, volume by mercury immersion and final moisture content.

APPENDIX B

PLOTS OF DEVIATOR STRESSES vs AXIAL COMPRESSIVE STRAIN

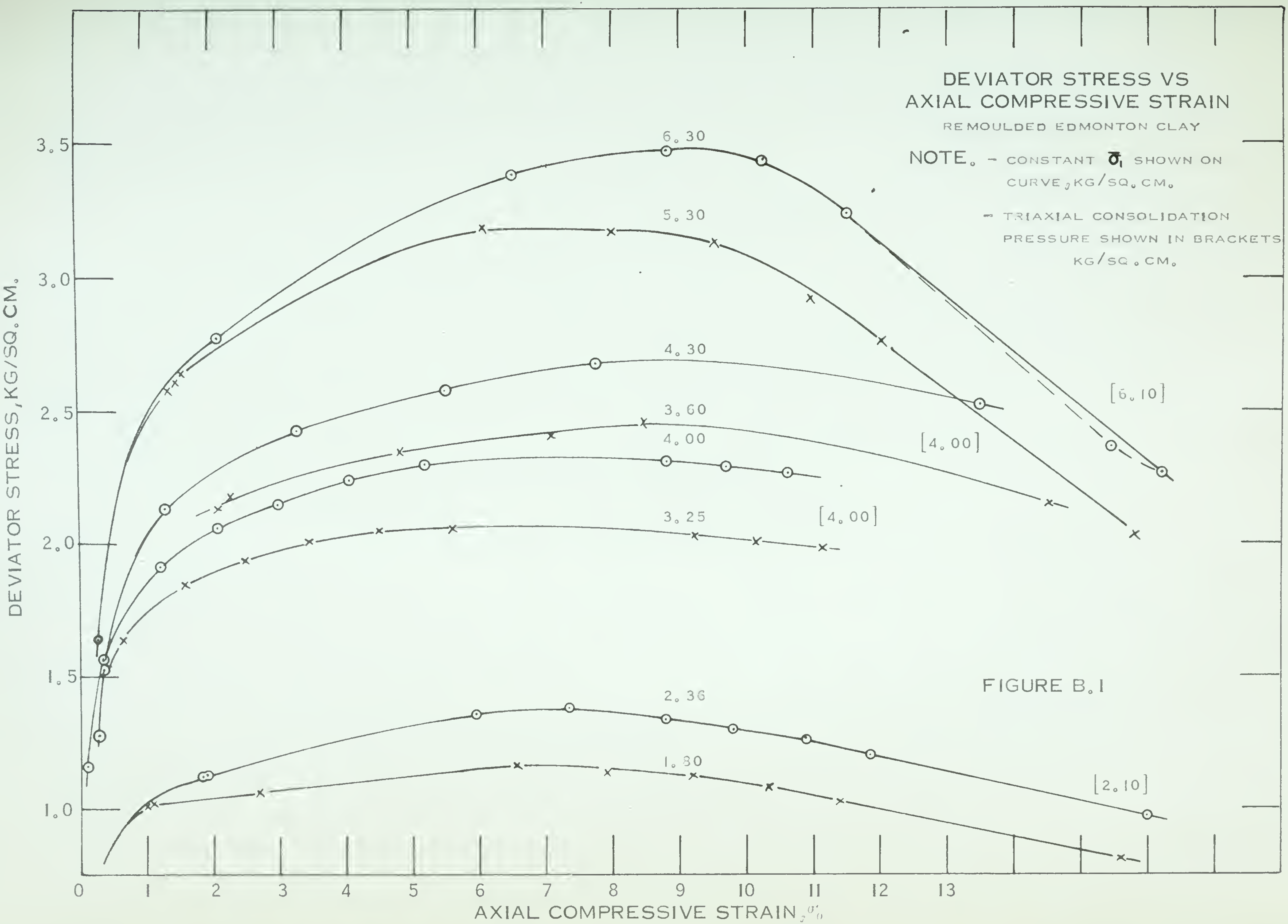

DEVIATOR STRESS VS
AXIAL COMPRESSIVE STRAIN
REMOULDED EDMONTON CLAY
NOTE. - CONSTANT σ̄1 SHOWN ON CURVE, KG/SQ.CM.
- TRIAXIAL CONSOLIDATION PRESSURE SHOWN IN BRACKETS KG/SQ.CM.
6.30
5.30
4.30
3.60
4.00
3.25
2.36
1.80
[6.10]
[4.00]
[4.00]
[2.10]
DEVIATOR STRESS, KG/SQ.CM.
AXIAL COMPRESSIVE STRAIN, %
1.0
1.5
2.0
2.5
3.0
3.5
0
1
2
3
4
5
6
7
8
9
10
11
12
13

FIGURE B.1

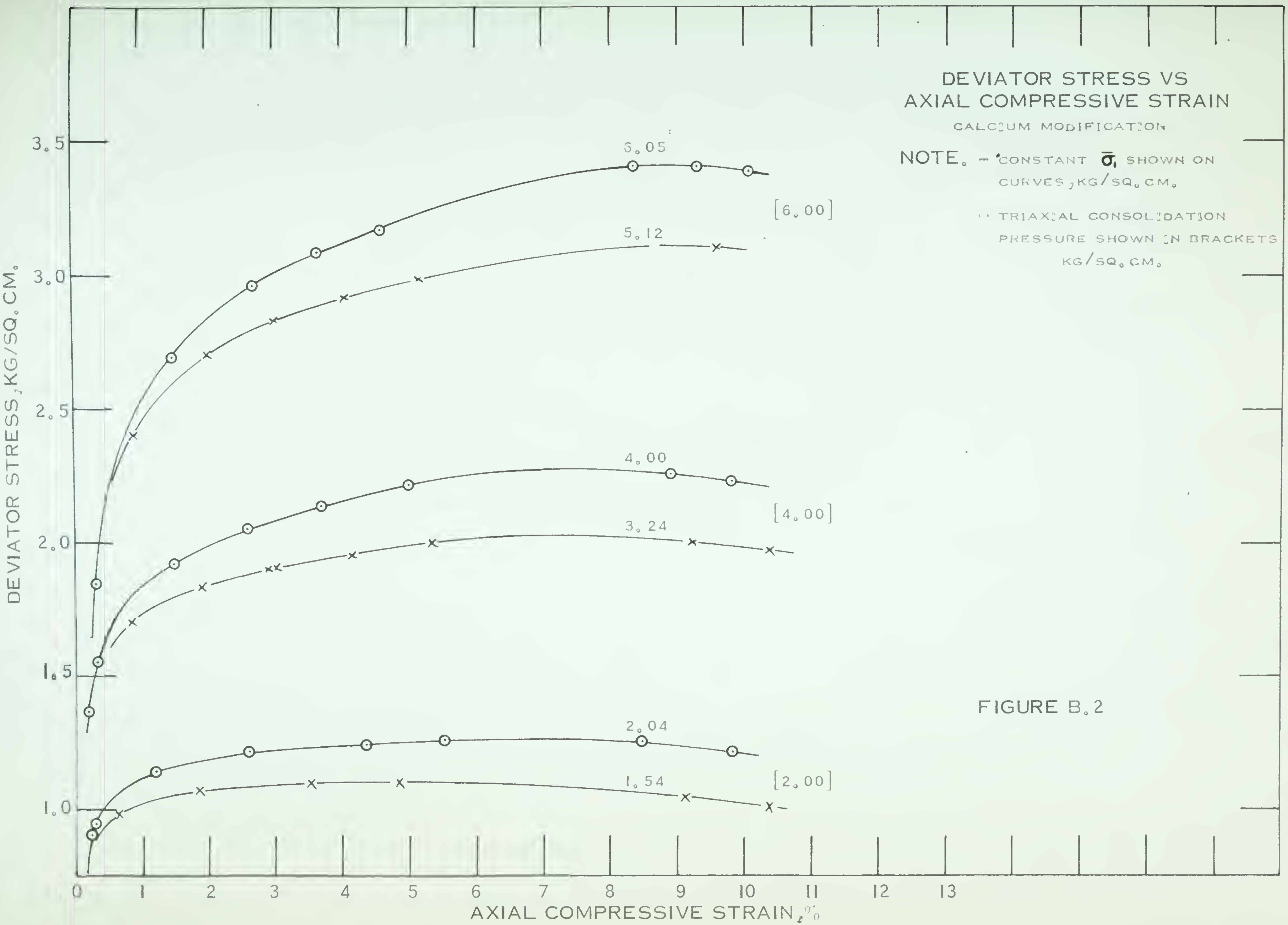

DEVIATOR STRESS VS
AXIAL COMPRESSIVE STRAIN
CALCIUM MODIFICATION
NOTE. - CONSTANT $\bar{\sigma}_1$ SHOWN ON CURVES, KG/SQ.CM.
TRIAXIAL CONSOLIDATION PRESSURE SHOWN IN BRACKETS KG/SQ.CM.
6.05
5.12
[6.00]
4.00
3.24
[4.00]
2.04
1.54
[2.00]
DEVIATOR STRESS, KG/SQ.CM.
3.5
3.0
2.5
2.0
1.5
1.0
AXIAL COMPRESSIVE STRAIN, %
0
1
2
3
4
5
6
7
8
9
10
11
12
13

FIGURE B.2

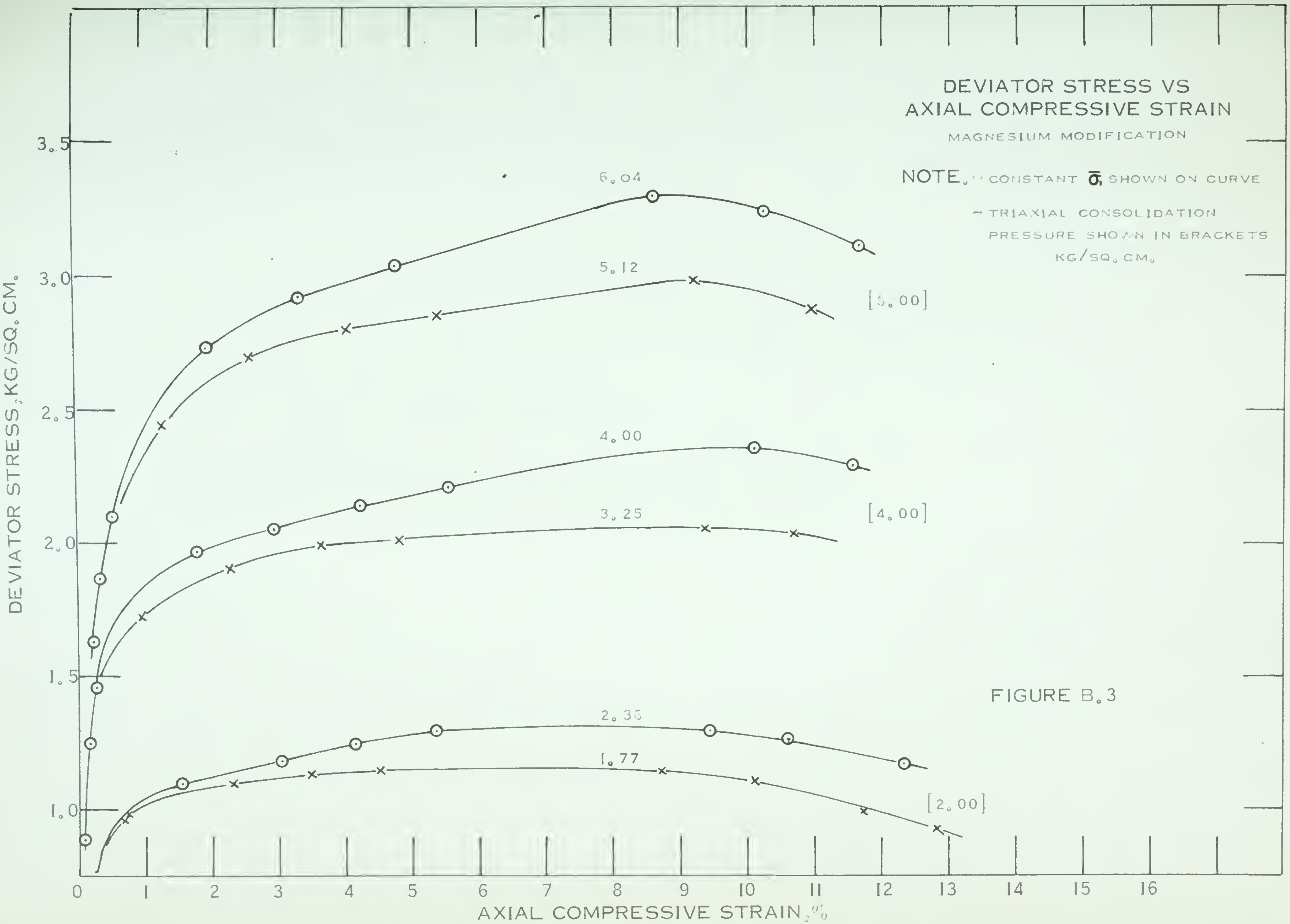

DEVIATOR STRESS VS
AXIAL COMPRESSIVE STRAIN
MAGNESIUM MODIFICATION
NOTE. — CONSTANT $\bar{\sigma}_1$ SHOWN ON CURVE
— TRIAXIAL CONSOLIDATION PRESSURE SHOWN IN BRACKETS KG/SQ.CM.
6.04
5.12
[6.00]
4.00
3.25
[4.00]
2.36
1.77
[2.00]
DEVIATOR STRESS, KG/SQ.CM.
AXIAL COMPRESSIVE STRAIN, %
3.5
3.0
2.5
2.0
1.5
1.0
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16

FIGURE B.3

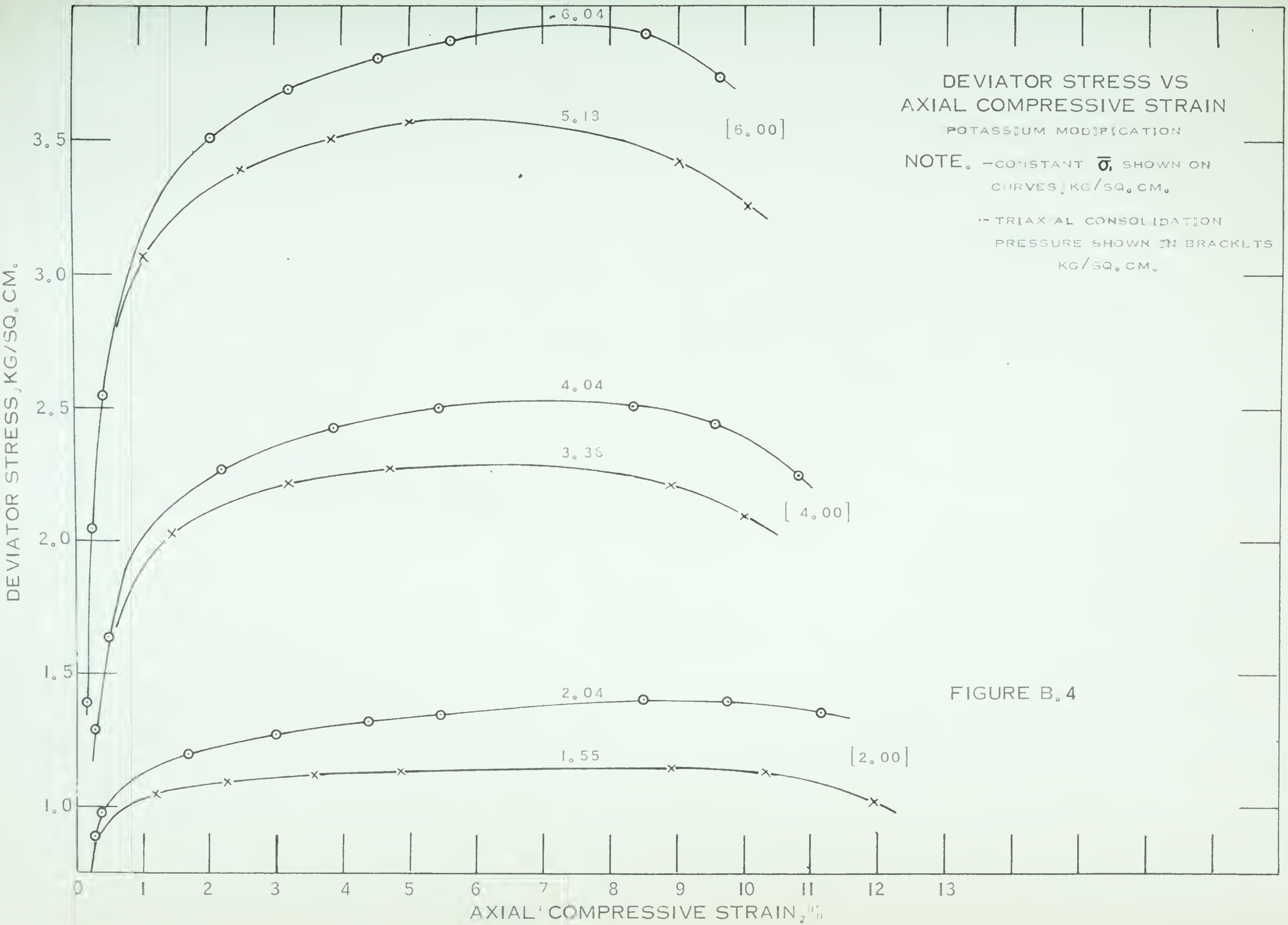

DEVIATOR STRESS VS
AXIAL COMPRESSIVE STRAIN
POTASSIUM MODIFICATION
NOTE. —CONSTANT $\overline{\sigma}_1$ SHOWN ON CURVES, KG/SQ.CM.
— TRIAXIAL CONSOLIDATION PRESSURE SHOWN IN BRACKETS KG/SQ.CM.
6.04
5.13
[6.00]
4.04
3.38
[4.00]
2.04
1.55
[2.00]
3.5
3.0
2.5
2.0
1.5
1.0
DEVIATOR STRESS, KG/SQ.CM.
0
1
2
3
4
5
6
7
8
9
10
11
12
13
AXIAL COMPRESSIVE STRAIN, %

FIGURE B.4

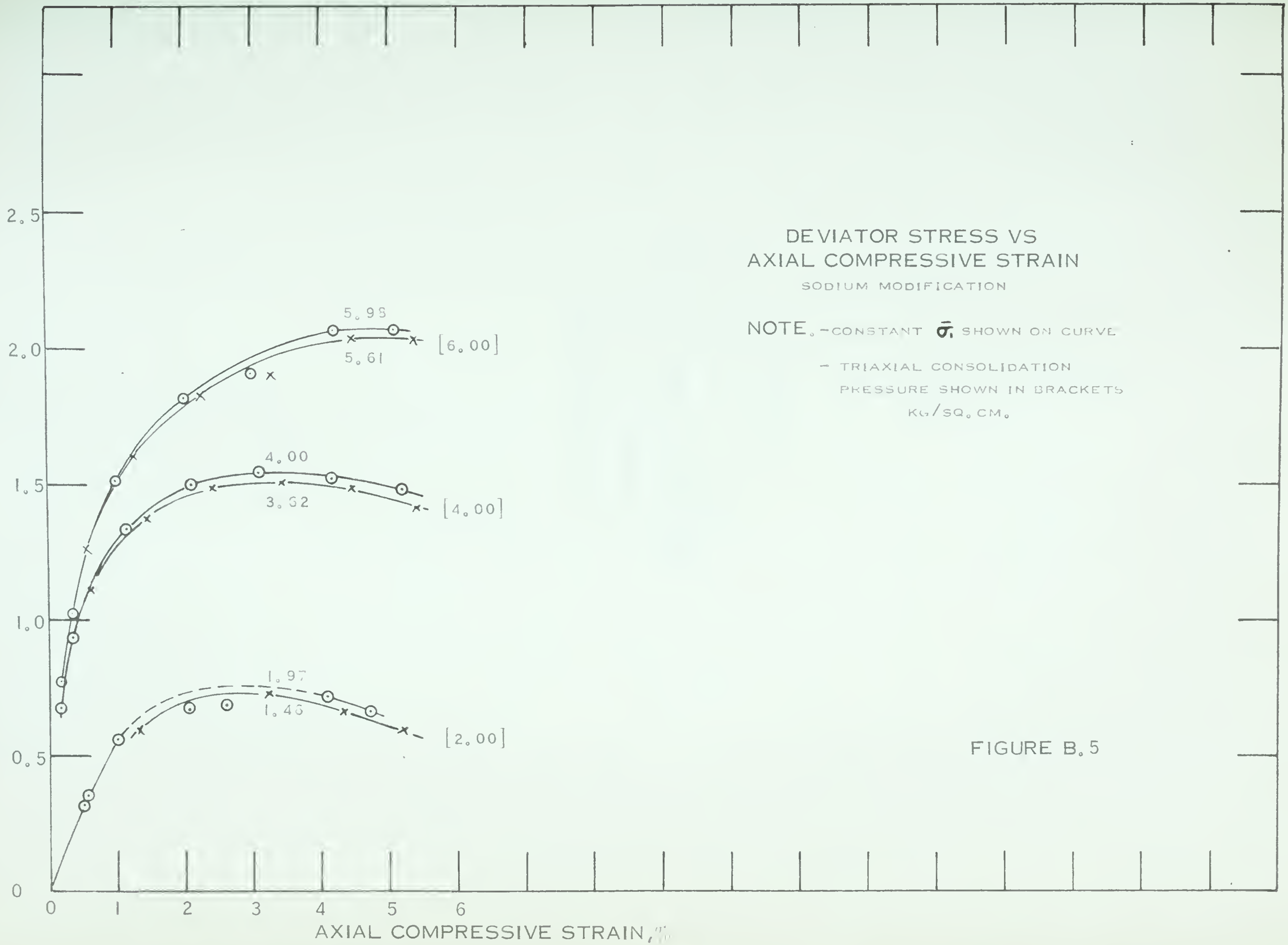

DEVIATOR STRESS VS
AXIAL COMPRESSIVE STRAIN
SODIUM MODIFICATION
NOTE. - CONSTANT $\bar{\sigma}_1$ SHOWN ON CURVE
- TRIAXIAL CONSOLIDATION
PRESSURE SHOWN IN BRACKETS
KG/SQ.CM.
5.98
5.61
[6.00]
4.00
3.62
[4.00]
1.97
1.46
[2.00]
2.5
2.0
1.5
1.0
0.5
0
0
1
2
3
4
5
6
AXIAL COMPRESSIVE STRAIN, %

FIGURE B.5

APPENDIX C

REPRESENTATIVE CFS TEST DATA

SHEETS AND SAMPLE CALCULATIONS

-REMOULDED LAKE EDMONTON CLAY

-SODIUM MODIFICATION

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Sample Description Removlded Edmonton Clay

146.02

Foil 1.10

Initial Weight 144.92 gms

Project THESIS

Hole No.

Depth sample NAT No.4

Engineer

Technician

Date of Set-up SEPT. 7, 1964

Test Lateral Pressure 4.00 KG/CM²

	1.	2.	Aver:
Length, mm	80.36	80.34	80.35
Diam. mm Top	36.30	36.30	36.30
Centre	36.26	36.28	36.27
Bottom	36.14	36.20	36.17

Area Top A_T 1034.9 sq.mm

Area Centre A_C 1033.2 sq.mm

Area Bottom A_B 1027.5 sq.mm

Average X-Sect Area

$= \frac{1}{4}(A_T + 2A_C + A_B)$

$= 10.322\ cm^2$

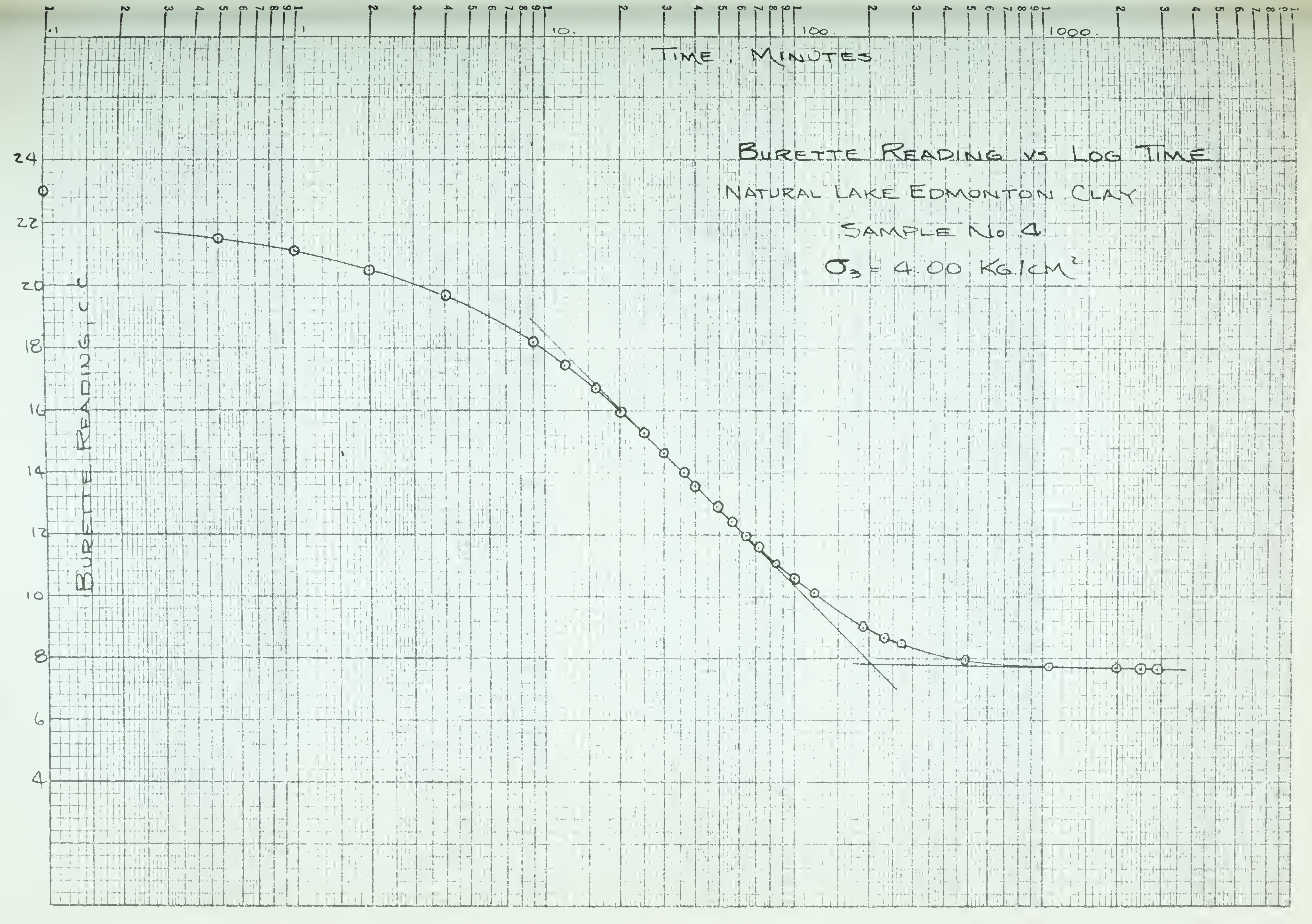
Original Volume 82.94 cc

CONSOLIDATION DATA

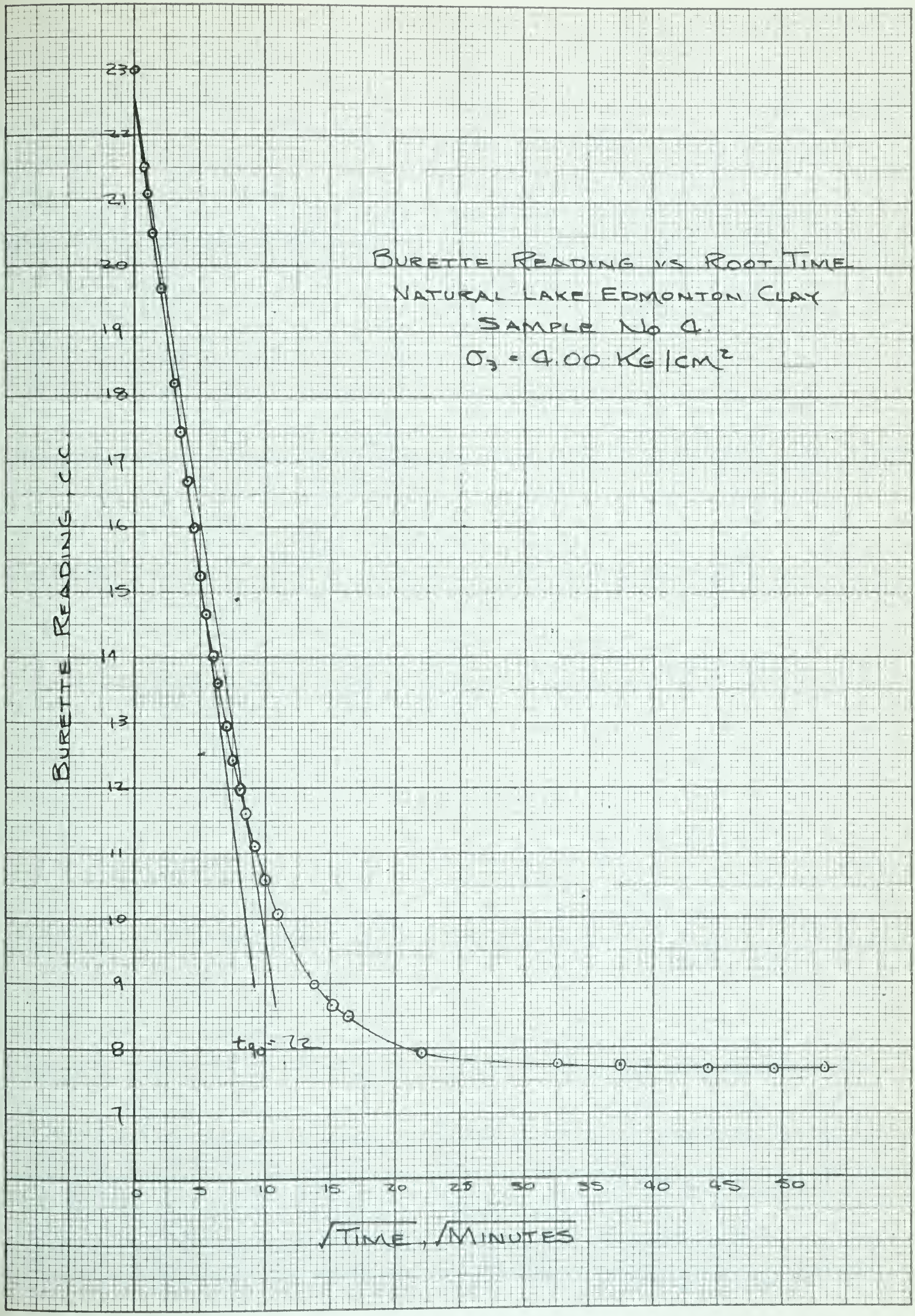
Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.	$\sqrt{\Delta t}$
7 1540	0	23.00		0
	.5	21.50		.71
	1.0	21.10		1.0
	2.0	20.50		1.4
	4.0	19.65		2.0
	9.0	18.20		3.0
	12.0	17.45		3.46
	16	16.70		4.0
	20	15.98		4.47
	25	15.23		5.0
	30	14.65		5.48
	36	14.00		6.0
	40	13.60		6.3
	49	12.92		7.0
	56	12.42		7.5
	64	11.92		8.0
	72	11.60		8.5
	84	11.10		9.16
	100	10.60		10.0
	120	10.07		11.0

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.	$\sqrt{\Delta t}$
1 1850	190	9.00		13.8
	230	8.68		15.2
2010	270	8.50		16.4
2350	490	7.92		22.1
Sept 8 0920	1060	7.74		32.6
[illegible]	[illegible]	7.71		37.4
Sept 9 0020	1960	7.68		44.3
0820	2440	7.65		49.4
1500	2840	7.64		53.3
		$\Delta V = 15.36$		
		K = 1.158		



BURETTE READING vs LOG TIME
NATURAL LAKE EDMONTON CLAY
SAMPLE No. 4
σ_3 = 4.00 KG/CM²
TIME, MINUTES
BURETTE READING, c.c.
.1
1
10.
100.
1000.
24
22
20
18
16
14
12
10
8
6
4



BURETTE READING vs ROOT TIME
NATURAL LAKE EDMONTON CLAY
SAMPLE No 4
σ_3 = 4.00 KG/CM²
BURETTE READING, C.C.
23
22
21
20
19
18
17
16
15
14
13
12
11
10
9
8
7
t_{90} = 72
0
5
10
15
20
25
30
35
40
45
50
$\sqrt{\text{TIME}}$, $\sqrt{\text{MINUTES}}$

Sheet 1

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

AXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No.

Depth Sample NAT No.4

Engineer [illegible]

Technician

Date of Test SEPT 10, 1964

Test Lateral Pressure σ_3 4.00 KG/CM²

Back Pressure 2.00 KG/CM²

Remarks

a at Failure

$(\sigma_1 - \sigma_3) = 2.33\ (2.085)$ $\sigma_1 = 6.33\ (6.06)$

2.33 (2.81)

= 4.00 (3.25)

= 1.67 (1.165)

in 7.0 (6.7) %

Area Correction Factor 1.158

1.158 × .0784 = .0908

+ve value (diff) rep. increase in vol. of spec.

Time min	Strain Dial Div. $L_0 = 80.35$	A_c cm²	No. of Stress Dial Div.		ΔV_{01} ~~Proving Ring Const~~ ~~δ kg/Div~~	c.c.	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot k}{A_c}$	Pore Press kg/cm² P_p	Effective Stress $\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor	Stress Ratio $\bar{\sigma}_1 / \bar{\sigma}_3$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_p}{\sigma_1 - \sigma_3}$
0	0	10.322	1168	0	1.80	0	0	0	4.00	4.00	1	0	--
6	8	10.332	1045	125	1.58	+.22	1.10	1.16	3.94			.1	
24	12	10.338	1036	132	1.57	+.23	1.16	1.16	4.00	2.84	1.41	.15	
40	20.1	10.348	1008	160	1.42	+.38	1.40	1.50	3.90			.25	
48	24.1	10.353	999	169	1.42	+.38	1.48	1.47	4.01			.30	
56	28.1	10.358	990	178	1.39	+.41	1.56	1.58	3.98			.35	
60	30.1	10.361	989	119	1.39	+.41	1.57	1.57	4.00	2.43	1.65	.375	
87	44.2	10.379	990	178	0.70	+1.10	1.56	2.48	3.08			.55	
15	48.2	10.384	987	181	0.70	+1.10	1.58	2.41	3.17			.60	
03	52.2	10.390	982	186	0.69	+1.11	1.63	2.38	3.25	1.62	2.01	.65	
55	80.4	10.426	962	206	1.10	+0.70	1.79	1.87	3.92			1.00	
88	96.4	10.447	948	220	1.12	+0.68	1.91	1.92				1.20	
92	984	10.450	947	221	1.12	+0.68	1.92	1.92	4.08	2.08	1.92	1.225	
36	120.5	10.479	958	210	0.50	+1.30	1.82	2.57	3.25			1.50	
52	128.6	10.489	954	214	0.43	+1.37	1.85	2.60	3.25	1.40	2.32	1.60	
20	164.7	10.538	934	234	1.00	+.80	2.02	2.06	3.96			2.05	
28	168.7	10.543	929	239	1.00	+.80	2.06	2.06	4.00	1.94	2.06	2.10	
84	196.9	10.581	943	225	0.32	+1.48	1.93	2.69	3.24			2.45	
91	200.9	10.587	942	226	0.35	+1.45	1.94	2.69	3.25	1.31	2.48	2.50	
54	233.0	10.630	923	245	0.92	0.88	2.09	2.14	3.95			2.90	
62	237.0	10.636	918	250	0.92	0.88	2.13^{5}	2.14	3.99^{5}			2.95	
70	241.1	10.641	916	252	0.92	0.88	2.15	2.15	4.00	1.85	2.16	3.00	
30	273.2	10.685	932	236	0.25	1.55	2.01	2.74	3.27			3.40	
38	277.2	10.691	931	237	0.25	1.55	2.01	2.76	3.25	1.24	2.62	3.45	
25	321.4	10.752	905	263	0.82	0.98	2.22	2.23	3.99			4.00	
33	325.4	10.758	903	265	0.82	+0.98	2.24	2.24	4.00	1.76	2.27	4.05	
94	357.6	10.803	927	241	0.19	+1.61	2.03	2.80	3.23			4.45	
02	361.6	10.808	924	244	0.20	+1.60	2.05	2.80	3.25	1.20	2.71	4.50	

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THRSIS

Hole No.

Depth

Sample NAT No4

Area Correction Factor 1.158

Time min	Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	ΔVol				$\sigma_1 - \sigma_3$ $= \frac{k_p \cdot \delta \cdot k}{}$	Pore Press kg/cm² P_f	Effective Stress $\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor	Stress Ratio $\bar{\sigma}_1 / \bar{\sigma}_3$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_f}{\sigma_1 - \sigma_3}$
	L_0 = 80.35	A_0 = 10.322	1168		.80	c.c.								
87	405.8	10.871	899	269	0.80	1.00		2.25	2.30	3.95			5.05	
310	417.8	10.888	892	276	0.80	1.00	—	2.30	2.30	4.00	1.70	2.35	5.20	
357	441.9	10.923	922	246	0.20	1.60		2.04	2.80	3.24			5.50	
365	445.9	10.928	921	247	0.20	1.60		2.05	2.80	3.25			5.55	
373	450.0	10.934	920	248	0.20	1.60	—	2.06	2.81	3.25	1.19	2.~~36~~73	5.60	
											cell P. fell slightly			
309	674.9	11.269	906	262	0.39	1.41		2.11	2.44	3.67			8.90	
373	707.1	11.318	880	288	0.80	1.00	—	2.31	2.31	4.00	1.69	2.37	8.80	
434	739.2	11.368	914	254	0.22	1.58	—	2.03	2.78	3.25	1.22	2.66	9.20	
503	775.4	11.424	882	286	0.82	0.98		2.27	2.28	3.99			9.65	
511	779.4	11.431	880	288	0.82	0.98	—	2.29	2.29	4.00	1.71	2.34	9.70	
582	815.6	11.488	914	254	0.24	1.56	—	2.01	2.76	3.25	1.24	2.62	10.15	
650	851.7	11.545	880	288	0.82	0.98	—	2.26 7*	2.27	4.00	1.73	2.31	10.60	
728	891.9	11.611	91[illegible]	253	0.30	1.50	—	1.98	2.73	3.25	1.27	2.56	11.10	

Moisture Content – Initial —

Final —

Degree of Saturation – Initial 97.5

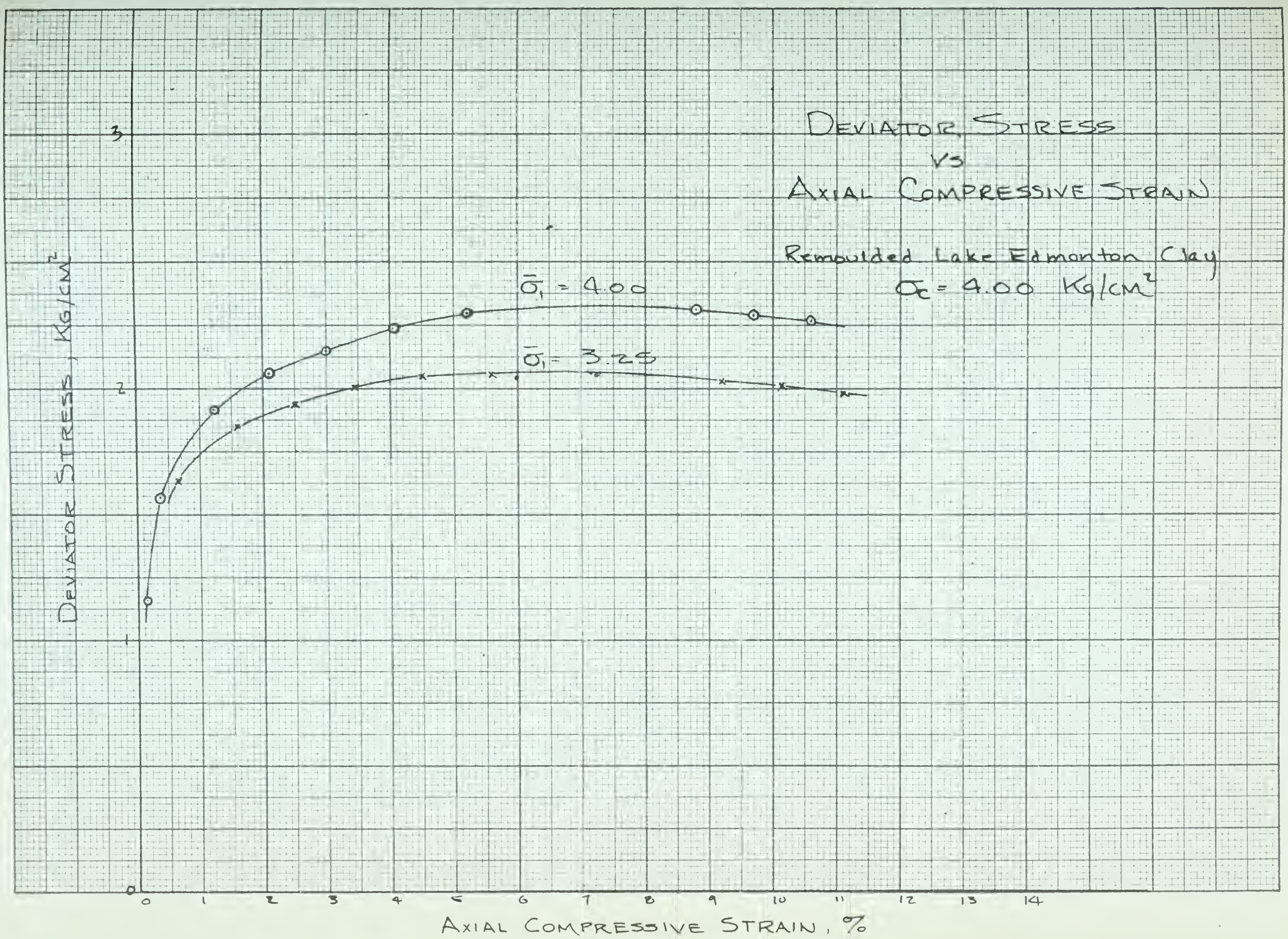
Final 48.0

Void Ratio – Initial —

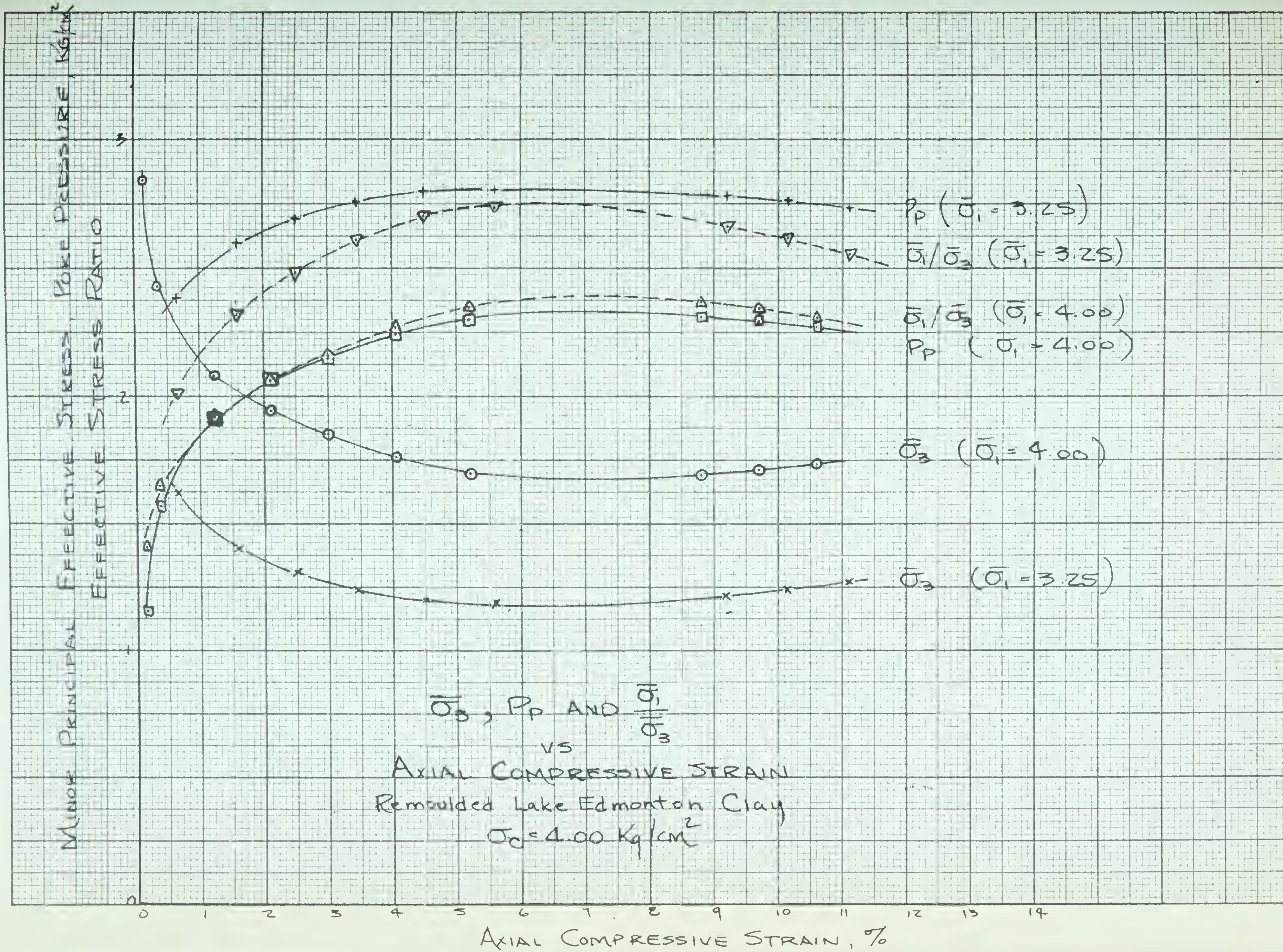
Final —

Pore Pressure Reaction 81

Sketch of Failure



DEVIATOR STRESS
VS
AXIAL COMPRESSIVE STRAIN
Remoulded Lake Edmonton Clay
$\sigma_c = 4.00$ Kg/cm^2
$\bar{\sigma}_1 = 4.00$
$\bar{\sigma}_1 = 3.25$
DEVIATOR STRESS, KG/CM^2
0
1
2
3
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14
AXIAL COMPRESSIVE STRAIN, %



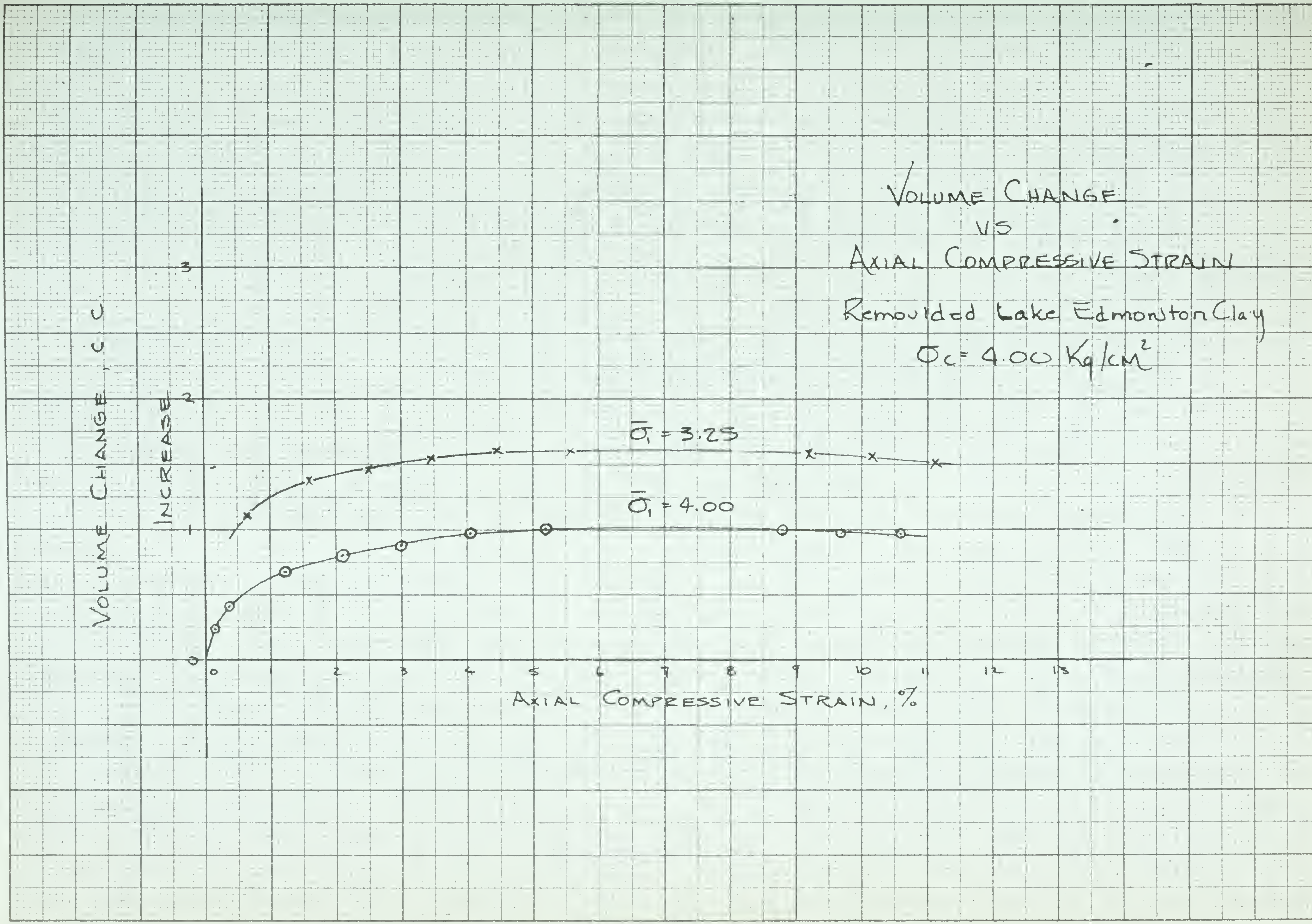
$\bar{\sigma}_3$, P_p AND $\dfrac{\bar{\sigma}_1}{\bar{\sigma}_3}$

VS

AXIAL COMPRESSIVE STRAIN

Remoulded Lake Edmonton Clay

$\sigma_c = 4.00$ Kg/cm²

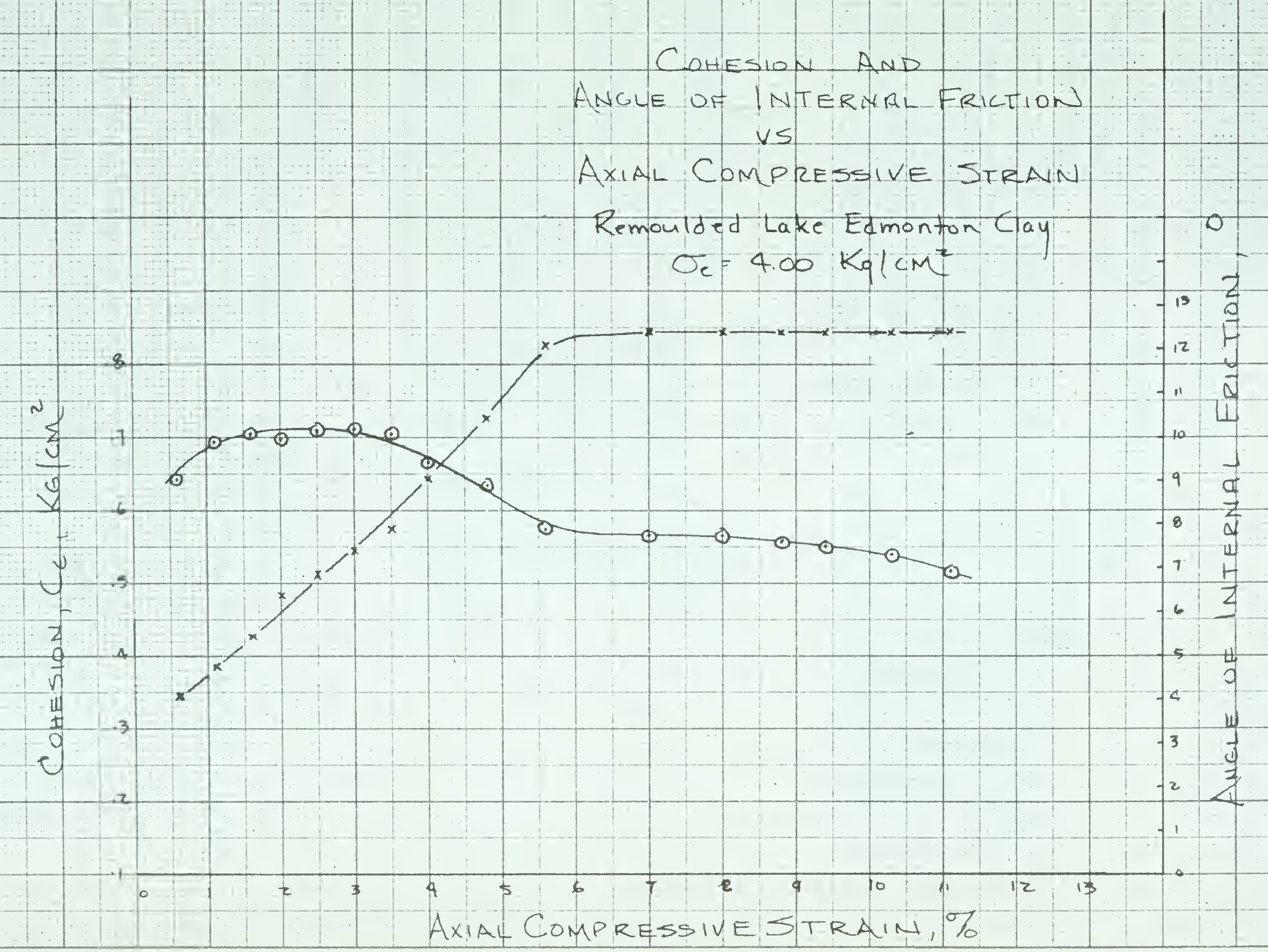


Volume Change
vs
Axial Compressive Strain
Remoulded Lake Edmonton Clay
σc = 4.00 Kg/cm²
σ̄1 = 3.25
σ̄1 = 4.00
Volume Change, c c
Increase
0
1
2
3
0
2
3
4
5
6
7
8
9
10
11
12
13
Axial Compressive Strain, %

Calculation of ϕ_ϵ and C_ϵ

Test No. 13 Sample. NAT. No 4 σ_c = 4.00 Kg/cm² Date. Sept 10.

Curve No.	ϵ %	$\bar{\sigma}_1$ Kg/cm²	$\Delta\bar{\sigma}_1$	σ_d Kg/cm²	$\frac{\sigma_d}{2}$	$\Delta\sigma_d$	$\sin\phi_\epsilon = \frac{\Delta\sigma_d}{2\Delta\bar{\sigma}_1 - \Delta\sigma_d}$	ϕ_ϵ	$\cos\phi_\epsilon$	$C_\epsilon = \frac{\frac{\sigma_d}{2} - (\bar{\sigma}_1 - \frac{\sigma_d}{2})\sin\phi_\epsilon}{\cos\phi_\epsilon}$
	0.6	4.00	.75	1.72	.86	0.10	0714	4°06'	.9974	.642
		3.25		1.62						
	1.10	4.00	.75	1.885	.943	0.115	.0830	4°46'	9965	.695
		3.25		1.77						
	1.6	4.00	.75	1.98	.99	0.13	.0949	5°22'	.9955	.703
		3.25		1.85						
	2.0	4.00	.75	2.045	1.023	0.15	.1111	6°23'	.9938	.697
		3.25		1.895						
	2.5	4.00	.75	2.10	1.05	0.16	.1194	6°52'	.9928	.705
		3.25		1.94						
	3.0	4.00	.75	2.15	1.075	0.17	.1278	7°21'	.9918	.711
		3.25		1.98						
	3.5	4.00	.75	2.195	1.098	0.18	.1364	7°51'	.9906	.705
		3.25		2.015						
	4.0	4.00	.75	2.235	1.118	0.205	.1583	9°06'	.9874	.666
		3.25		2.03						
	4.8	4.00	.75	2.285	1.143	0.23	.1811	10°26'	.9835	.633
		3.25		2.055						
	5.6	4.00	.75	2.32	1.16	0.260	.2097	12°06'	.9778	.573
		3.25		2.06						
	7.0	4.00	.75	2.33	1.165	0.265	.2146	12°24'	.9767	.568
		3.25		2.065						
	8.0	4.00	.75	2.325	1.163	0.265	.2146	12°24'	.9767	.566
		3.25		2.06						
	8.8	4.00	.75	2.31	1.155	0.265	.2146	12°24'	.9767	.558
		3.25		2.045						
	9.4	4.00	.75	2.295	1.148	0.265	.2146	12°24'	.9767	.551
		3.25		2.03						
	10.3	4.00	.75	2.275	1.137	0.265	.2146	12°24'	.9767	.540
		3.25		2.01						
	11.1	4.00	.75	2.245	1.123	0.265	.2146	12°24'	.9767	.515
		3.25		1.98						


COHESION AND
ANGLE OF INTERNAL FRICTION
VS
AXIAL COMPRESSIVE STRAIN
Remoulded Lake Edmonton Clay
σ_c = 4.00 Kg/cm²
COHESION, C_e, KG/CM²
ANGLE OF INTERNAL FRICTION, Ø
AXIAL COMPRESSIVE STRAIN, %

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING

SOIL MECHANICS LABORATORY
TRIAXIAL COMPRESSION TEST
PORE PRESSURE REACTION TEST

Sample No 4
Sample Desc. NAT.
Kg/sq. cm on cell 4.00

Backpressure - 2 Kg/cm²
"Load" - 1 Kg/cm²

"LOAD" t min	Pp Kg/cm^2	"UNLOAD" t min	Pp Kg/cm^2
0	0	0	.81
0.1	–	0.1	.50
0.25	.35	0.25	.40
0.50	.48	0.50	.32
1.0	.62	1.0	.22
2.	.72	2.	.13
3	.78	3	.09
4	.79	4	.07
5	.81	5	.05

FINAL WET WT 134.08

Final Moisture Content

		INITIAL
Wet Wt. plus tare	163.34	73.38
Dry Wt. plus tare	127.69	59.57
Wt. of water	3565	13.81
Wt. and No. of tare	A-12 30.32	A-9 30.33
Wt. of dry soil	97.37	29.24
Final M.C.	36.61	47.23

Final Volume

Wt. Hg + Tare	438.32	705.83
Tare	84.48	84.49
Wt. Hg	353.84	621.34
Temp.	24.6	
Vol. Hg	975.13/13.5352 = 72.05	

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING

SOIL MECHANICS LABORATORY
TRIAXIAL COMPRESSION TEST COMPUTATIONS

Sample No. 4
Sample Desc. NAT.

BEGINNING OF TEST

Original Vol of Specimen 82.94

Wt. of Soil Solids in Specimen 98.43

Vol. of Soil Solids 35.28

Vol of Voids 47.66

Original Void Ratio, e_o 1.35

Original Porosity, n, 0.57

Wt. of Water 46.49

Original Degree of Saturation, S, 97.5

Original Wet Wt., lb/cu. ft. 108.87

Original Dry Wt., lb/cu. ft. 73.94

END OF TEST

Final Vol of Specimen (by Hg immersion) 72.05

Vol. of Soil Solids 35.28

Vol. of Voids 36.77

Final Void Ratio, e_f 1.04

Final Porosity, n, 0.51

Wt. of Water from final m.c. 36.04

Final Degree of Saturation 98.0

Final Wet Wt. lb/cu.ft. 115.95

Final Dry Wt. lb/cu.ft. 85.12

Wet wt. of specimen at beginning of test 144.92 gm.

Wet wt. of specimen at end of test 134.08 gm.

Weight loss 10.84 gm.

Vol. change from burette rdg. discrepancy 4.57 cc. Vol. inc.

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

RIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No.

Depth sample Na No.2

Engineer [signature]

Technician

Date of Set-up AUG 21, 1964

Test Lateral Pressure 4.00 KG/CM^2

mple Description SODIUM MODIFIED EDMONTON CLAY

132.85

1.10

tial Weight 131.75 gms

th, mm	1. 80.20	2. 80.20	Aver: 80.20	Area Top	A_T 1036.6	sq.mm
. mm Top	1. 36.32	2. 36.34	Aver. 36.33	Area Centre	A_C 1036.0	sq.mm
Centre	1. 36.34	2. 36.30	Aver: 36.32	Area Bottom	A_B 1035.4	sq.mm
Bottom	1. 36.32	2. 36.30	Aver: 36.31			

nal Volume 83.087 cc

Average X–Sect Area

$= \frac{1}{4}(A_T + 2A_C + A_B)$

$= 10.360$

SOLIDATION DATA

e	Δt min.	Burette Rdg. c.c.	ΔV c.c.	$\sqrt{\Delta t}$
0900	0	24.00		0
	1	23.85		1
	4	23.65		2
	9	23.55		3
	16	23.46		4
	25	23.35		5
	36	23.28		6
	49	23.18		7
	64	23.10		8
	81	23.00		9
	100	22.90		10.
	121	22.82		11
	160	22.70		12.65
	260	22.40		16.1
	365	22.150		18.8
	600	21.71		24.5
	750	21.50		27.4
0820	1400	20.77		37.4
1140	1600	20.60		40.0
2200	2220	20.10		47.1

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.	$\sqrt{\Delta t}$
AUG 23 0920	2900	19.69		53.8
AUG 24 0840	4300	18.98		65.6
1700	4800	18.72		69.3
AUG 25 0800	5700	18.34		75.5
2100	6480	18.01		80.5
AUG 26 1040	7300	17.72		85.4
2310	8050	17.43		89.7
AUG 27 0820	8600	17.25		92.8
AUG 28 1640	10,540	16.60		102.7
AUG 29 1200	12,000	16.13		109.6
AUG 30 1200	13,140	15.82		114.6
AUG 31 1040	14,500	15.48		120.5
Sept 1 1140	16,000	15.10		126.5
Sept 2 1100	17,400	14.75		131.9
Sept 3 1340	19,000	14.40		137.9
Sept 5 0950	21,650	13.84		147.2
Sept 6 1100	23,160	13.55		152.2
Sept 7 1740	25,000	13.22		158.1
Sept 8 1020	26,000	13.04		161.3
Sept 10 1220	29,000	12.57		170.3

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

RIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No.

Depth sample Na No 2

Engineer

Technician

Date of Set-up AUG 21 /64

Test Lateral Pressure 4.00

ample Description

itial Weight gms

gth, mm 1. 2. Aver: Area Top A_T Sq.mm

m. mm Top 1. 2. Aver. Area Centre A_C sq.mm

Centre 1. 2. Aver: Area Bottom A_B sq.mm

Bottom 1. 2. Aver: Average X-Sect Area

inal Volume cc

$= \frac{1}{4}(A_T + 2A_C + A_B)$

=

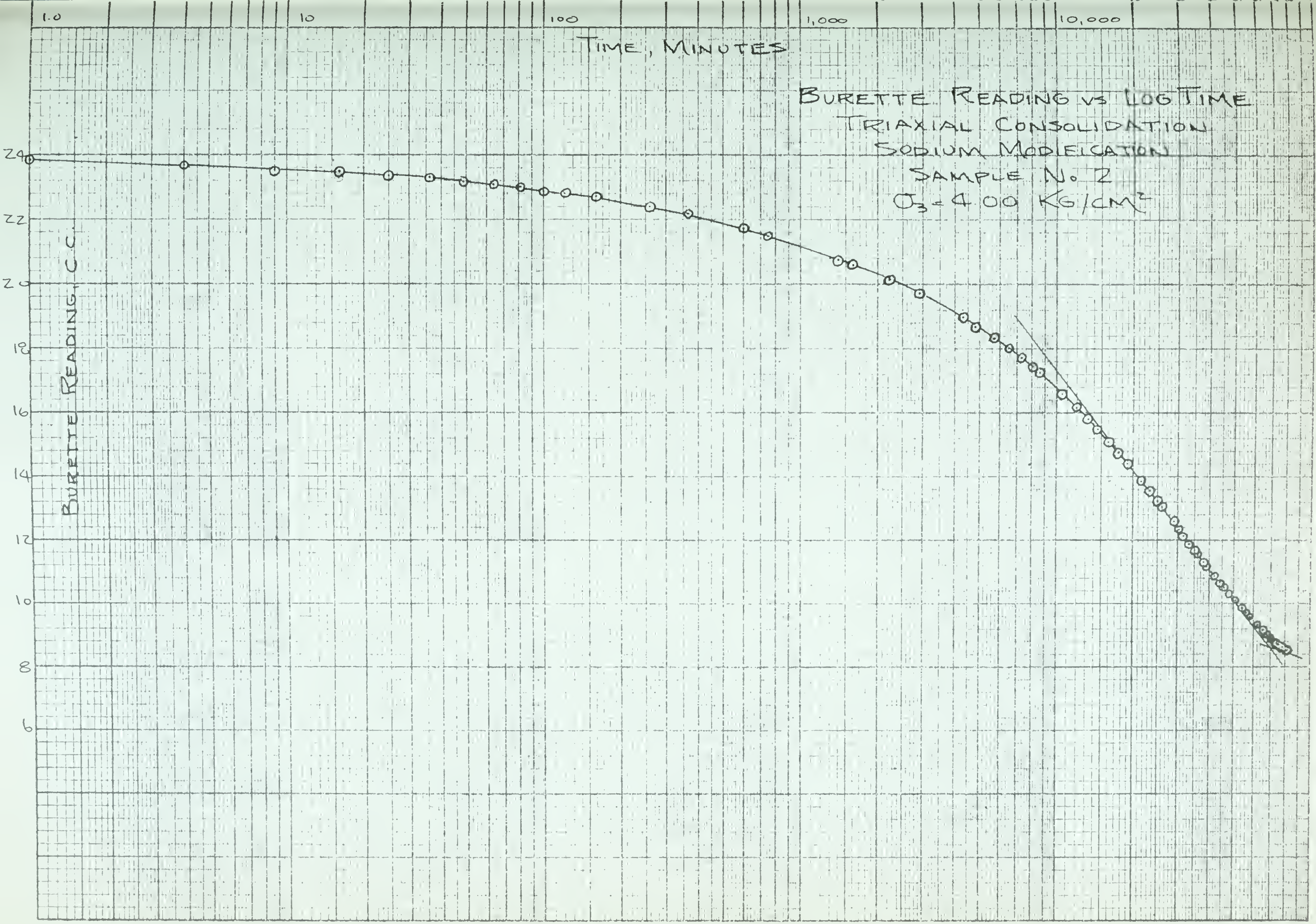
SOLIDATION DATA

CONTINUED

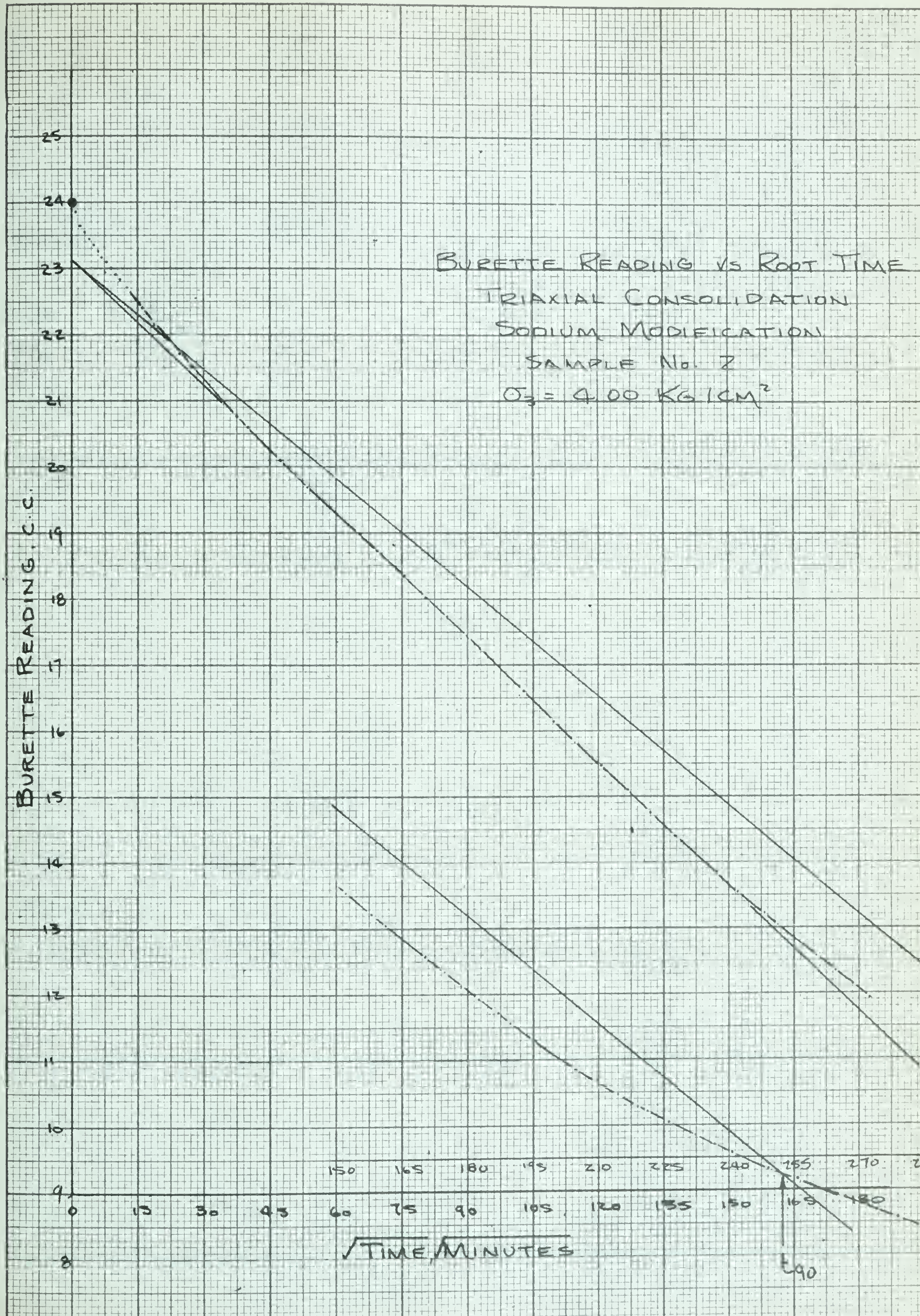
te / ne	Δ t min.	Burette Rdg. c.c.	~~Δ V c.c.~~ √Δt
11 1320	30,500	12.33	174.6
12 1150	31,850	12.13	178.5
13 1230	33,330	11.92	182.6
14 1620	35,000	11.71	187.0
15 0900	36,000	11.57	189.7
16 2000	38,100	11.30	195.2
17 1100	39,000	11.17	197.5
18 1300	42,000	10.85	205.0
20 2220	44,000	10.67	209.8
22 0740	46,000	10.47	214.5
23 1700	48,000	10.29	219.1
24 0940	49,000	10.21	221.3
25 1900	51,000	10.05	225.9
27 2100	54,000	9.89	232.3
29 0640	56,020	9.70	236.8
30 1540	58,000	9.57	240.8
3 1025	62,000	9.31	249.0
5 1220	65,000	9.18	255.5
2130	67,000	9.06	258.8
1420	68,000	9.01	260.8

CONSOLIDATION DATA – cont'd

Date Time	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
Oct 8 2340	70,000	8.90	264.6
Oct 9 1620	71,000	8.86	266.5
Oct 11 1820	74,000	8.74	272.0
Oct 12 1100	75,000	8.70	274.0
Oct 14 1300	78,000	8.59	279.0
Oct 16 1500	81,000	8.49	
	FINAL	ΔV = 15.51	
		K = 1.16	



Burette Reading vs Log Time
Triaxial Consolidation
Sodium Modification
Sample No. 2
σ_3 = 4.00 Kg/cm²
Time, Minutes
1.0
10
100
1,000
10,000
Burette Reading, C.C.
24
22
20
18
16
14
12
10
8
6



BURETTE READING VS ROOT TIME
TRIAXIAL CONSOLIDATION
SODIUM MODIFICATION
SAMPLE No. 2
σ_3 = 4.00 KG/CM^2
BURETTE READING, C.C.
25
24
23
22
21
20
19
18
17
16
15
14
13
12
11
10
9
8
0
15
30
45
60
75
90
105
120
135
150
165
180
150
165
180
195
210
225
240
255
270
285
$\sqrt{\text{TIME}}$, MINUTES
t_{90}

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

...IAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No.

Depth Sample Na. No 2

Engineer

Technician

Date of Test OCTOBER 18, 1964

Test Lateral Pressure σ_3 4.00 KG/CM²

Back Pressure 2.00 KG/CM²

Remarks

...at Failure

$(\sigma_1 - \sigma_3) =$ ~~0.15~~ 1.54 (1.51) $\sigma_1 =$ 5.55 (5.50)

1.55 (1.88)

4.00 (3.62)

2.44 (2.11)

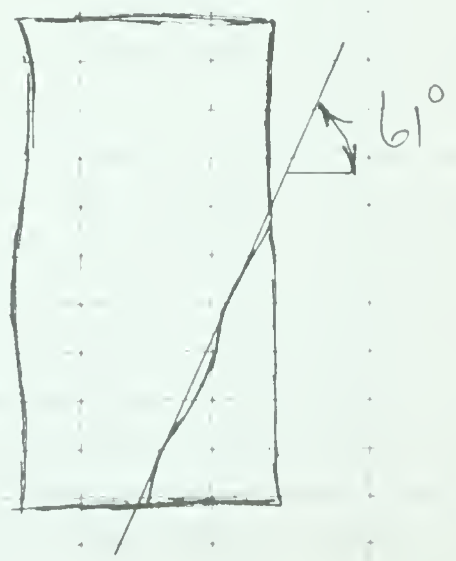
3.4 (3.4)

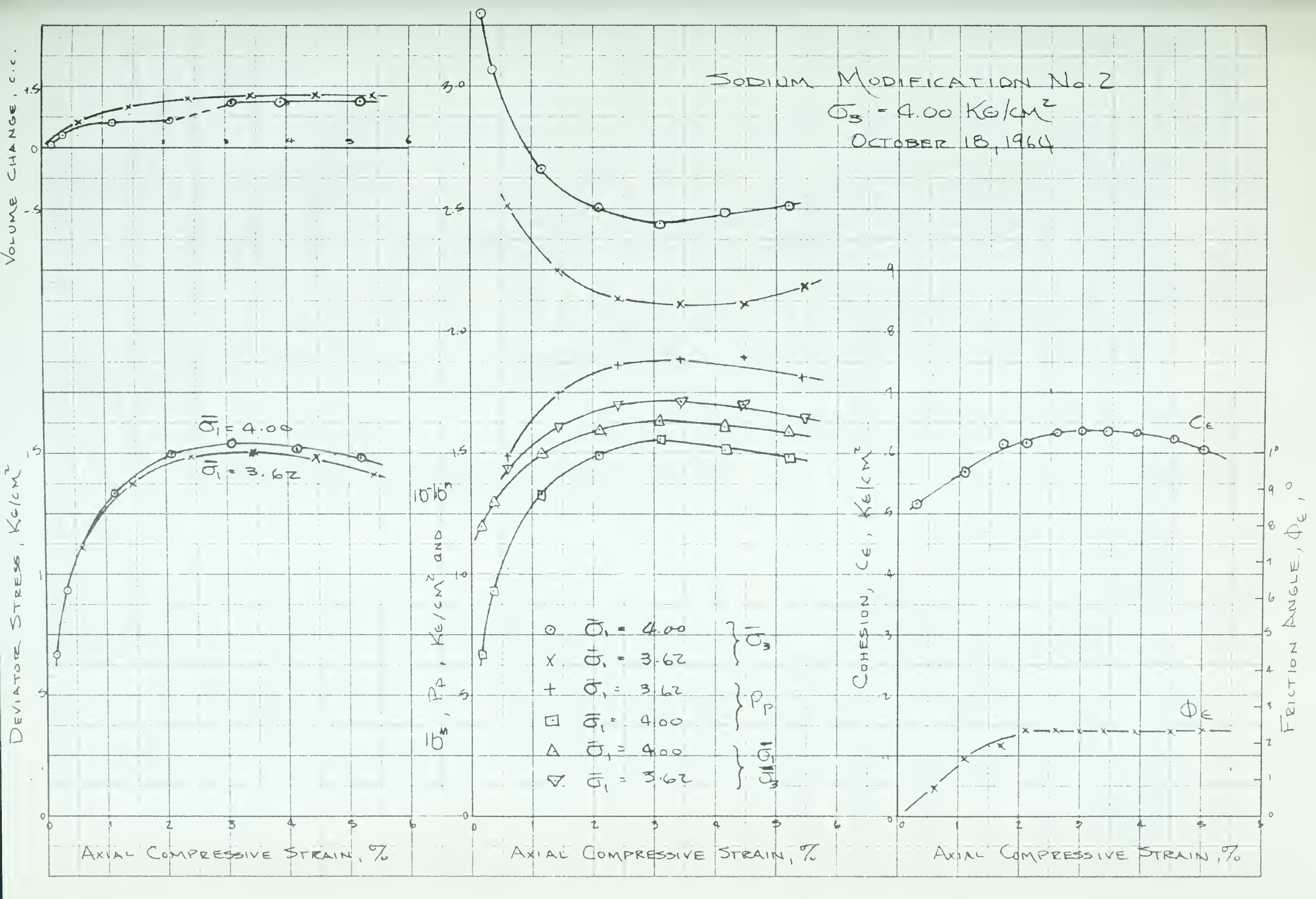
Area Correction Factor 1.160

·0784 x 1.160 = .0909

+ve = vol increase

...e ...n ...oo 18	Strain Dial Div. $L_0 = 80.20$	A_c cm²	No. of Stress Dial Div.		ΔVol. c.c.		$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot k}{}$	Pore Press kg/cm² □ +	Effective Stress $\bar{\sigma}_1$ Major ○	$\bar{\sigma}_3$ Minor x	Stress Ratio $\bar{\sigma}_1/\bar{\sigma}_3$ △ ▽	Axial Comp. Strain %	$\bar{A}$ $P_f/(\sigma_1-\sigma_3)$
0	0	10.360	1128	0	+0.20	0	0	0	4.00	4.00	1.00	0	
...09	12	10.376	1059	69	·21	+·01	0.60	0.69	3.91			15	
...43	14	10.378	1052	76	·22	·02	0.67	0.67	4.00	3.33	1.20	·175	
...33	28.1	10.396	1022	106	·30	·10	0.93	0.93	4.00	3.07	1.30	·35	
...34	48.1	10.423	1001	127	·40	·20	1.11	1.49	3.62	2.51	1.44	·60	
...80	92.2	10.481	975	153	·40	·20	1.33	1.33	4.00	2.67	1.50	1.15	
...8	114.3	10.510	969	159	·52	·32	1.38	1.75	3.63	2.25	1.61	1.425	
...20	168.4	10.582	953	175	·42	·22	1.50	1.49	4.01	2.51	1.60	2.10	
...47	192.5	10.615	954	174	+·60	+·40	1.49	1.87	3.62	2.13	1.70	2.40	
...30	248.6	10.691	947	181	·58	·38	1.54	1.56	3.98	2.44	1.63	3.10	
...65	274.7	10.727	952	176	·63	·43	1.51	1.88	3.63	2.12	1.71	3.425	
...95	332.8	10.809	947	181	·59	·39	1.52	1.51	4.01	2.49	1.61	4.15	
...40	358.9	10.845	949	179	·63	·43	1.50	1.90	3.60	2.10	1.71	4.475	
...52	417.0	10.928	950	178	·58	·38	1.48	1.49	3.99	2.51	1.59	5.20	
...30	433.1	10.951	958	170	+·61	+·41	1.41	1.81	3.60	2.19	1.64	5.40	





SODIUM MODIFICATION No. 2
σ_3 = 4.00 KG/CM^2
OCTOBER 18, 1964
VOLUME CHANGE, c.c.
DEVIATOR STRESS, KG/CM^2
$\bar{\sigma}_1$ = 4.00
$\bar{\sigma}_1$ = 3.62
$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$, P_P, KG/CM^2 AND
$\bar{\sigma}_1$ = 4.00
$\bar{\sigma}_1$ = 3.62
$\bar{\sigma}_1$ = 3.62
$\bar{\sigma}_1$ = 4.00
$\bar{\sigma}_1$ = 4.00
$\bar{\sigma}_1$ = 3.62
$\bar{\sigma}_3$
P_P
$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$
COHESION, C_E, KG/CM^2
FRICTION ANGLE, ϕ_E, °
C_E
ϕ_E
AXIAL COMPRESSIVE STRAIN, %
AXIAL COMPRESSIVE STRAIN, %
AXIAL COMPRESSIVE STRAIN, %

CALCULATION OF ϕ_ϵ AND C_ϵ

TEST No. 16 SAMPLE. Na Mod. No 2 σ_c = 4.00 KG/CM² DATE. Oct. 24

CURVE No.	ϵ %	$\bar{\sigma}_1$ KG/CM²	$\Delta\bar{\sigma}_1$	σ_d KG/CM²	$\frac{\sigma_d}{2}$	$\Delta\sigma_d$	$\sin\phi_\epsilon = \frac{\Delta\sigma_d}{2\Delta\bar{\sigma}_1 - \Delta\sigma_d}$	ϕ_ϵ	$\cos\phi_\epsilon$	$C_\epsilon = \frac{\frac{\sigma_d}{2} - (\bar{\sigma} - \frac{\sigma_d}{2})\sin\phi_\epsilon}{\cos\phi_\epsilon}$
	0.6	4.00 3.62	.38	1.12 1.11	.56 .555	.01	.0133	0°46′	.9999	.514
	1.1	4.00 3.62	.38	1.32 1.30	.66 .65	.02	.0270	01°33′	.9996	.570
	1.7	4.00 3.62	.38	1.45 1.425	.725 .7125	.025	.0340	01°57′	.9994	.614
	2.1	4.00 3.62	.38	1.50 1.47	.75 .735	.03	.04110	02°21′	.9992	.616
	2.6	4.00 3.62	.38	1.53 1.50	.765 .75	.03	.0411	02°21′	.9992	.632
	3.0	4.00 3.62	.38	1.54 1.51	.77 .755	.03	.0411	02°21′	.9992	.638
	3.4	4.00 3.62	.38	1.54 1.51	.77 .755	.03	.0411	02°21′	.9992	.638
	3.9	4.00 3.62	.38	1.53 1.50	.765 .75	.03	.0411	02°21′	.9992	.632
	4.5	4.00 3.62	.38	1.515 1.475	.7575 .7375	.03	.0411	02°21′	.9992	.625
	5.0	4.00 3.62	.38	1.48 1.45	.74 .725	.03	.0411	02°21′	.9992	.606

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING

SOIL MECHANICS LABORATORY
TRIAXIAL COMPRESSION TEST
PORE PRESSURE REACTION TEST

Sample No 2
Sample Desc. Na
Kg/sq. cm on cell 4.00

Back pressure = 2
Load = 1 Kg/cm²

"LOAD" t min	Pp Kg/cm^2	"UNLOAD" t min	Pp Kg/cm^2
0	0	0	0
0.1	-	0.1	.15
0.25	.10	0.25	.14
0.50	.13	0.50	.13
1.0	.15	1.0	.12
2.	.19	2.	.10
3	.21	3	.09
4	.23	4	.08
5	.24	5	.06

FINAL WET WT = 117.65

Final Moisture Content

Wet Wt. plus tare	167.72	62.71
Dry Wt. plus tare	132.11	48.88
Wt. of water	35.61	13.83
Wt. and No. of tare	A 68.30	A-5 30.13
Wt. of dry soil	63.81	18.75
Final M.C.	55.81	73.76

Final Volume

Wt. Hg + Tare	649.52	582.45
Tare	129.45	129.47
Wt. Hg	520.07	452.98
Temp.	25.4	

Vol. Hg 973.05 / 13.5331 = 71.90

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING

SOIL MECHANICS LABORATORY
TRIAXIAL COMPRESSION TEST COMPUTATIONS

Sample No. 2
Sample Desc. Na. Mod.

BEGINNING OF TEST

Original Vol of Specimen	83.09
Wt. of Soil Solids in Specimen	75.82
Vol. of Soil Solids	27.57
Vol of Voids	55.52
Original Void Ratio, e_o	2.014
Original Porosity, n,	0.668
Wt. of Water	55.92
Original Degree of Saturation, S,	100.7
Original Wet Wt., lb/cu. ft.	98.80
Original Dry Wt., lb/cu. ft.	56.86

END OF TEST

Final Vol of Specimen (by Hg immersion)	71.90
Vol. of Soil Solids	27.57
Vol. of Voids	44.33
Final Void Ratio, e_f	1.608
Final Porosity, n,	0.617
Wt. of Water	from final m.c. 42.32
Final Degree of Saturation	95.47
Final Wet Wt. lb/cu.ft.	101.96
Final Dry Wt. lb/cu.ft.	65.71
Wet wt. of specimen at beginning of test	131.75 gm.
Wet wt. of specimen at end of test	117.65 gm.
Weight loss	14.10 gm.
Vol. change from burette rdg. discrepancy	1.11 cc. Vol. Inc.

APPENDIX D

COMPOSITE TEST RESULTS

ADVANCED SOIL MECHANICS LABORATORY

UNIVERSITY OF ALBERTA, EDMONTON

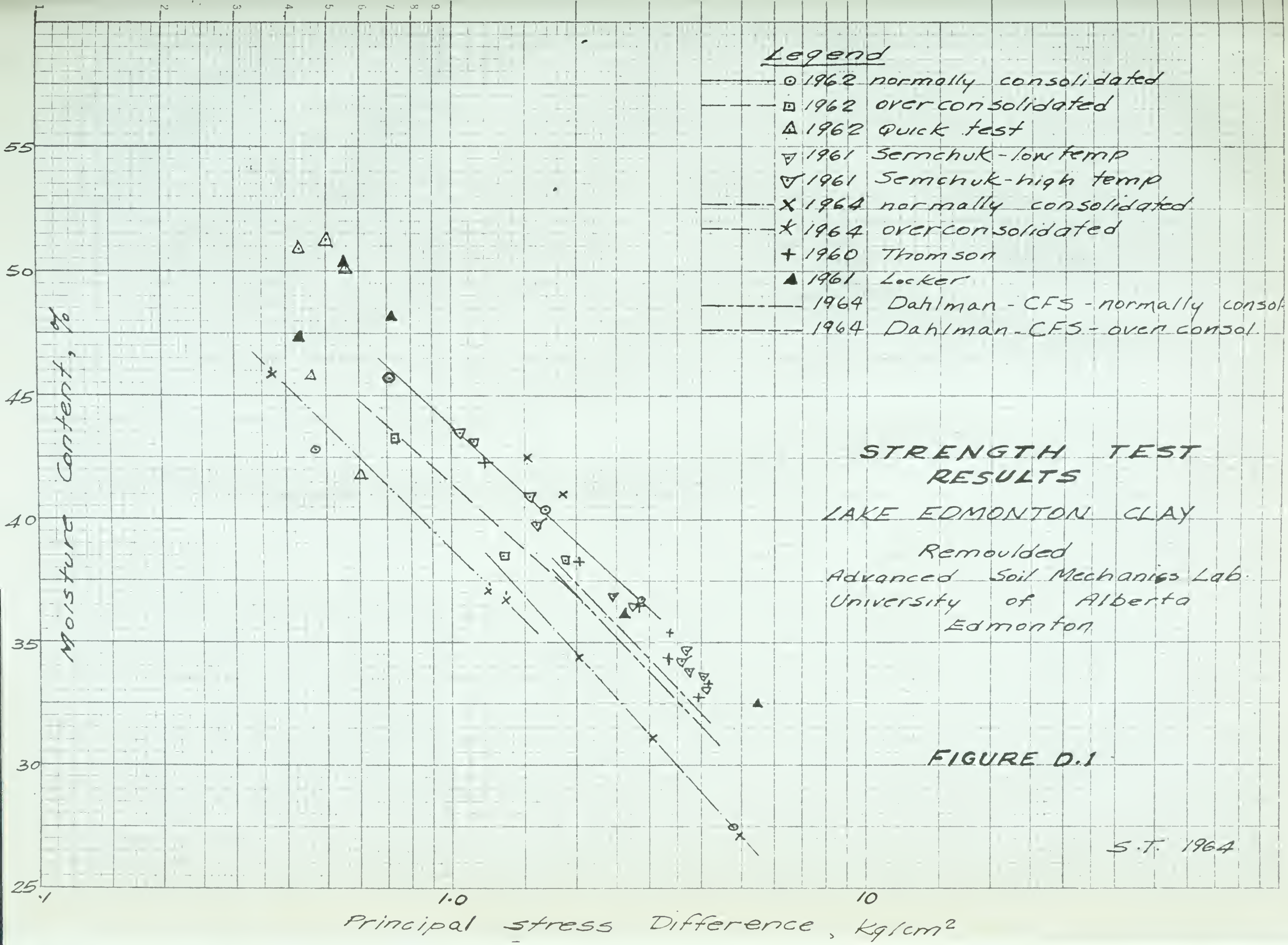
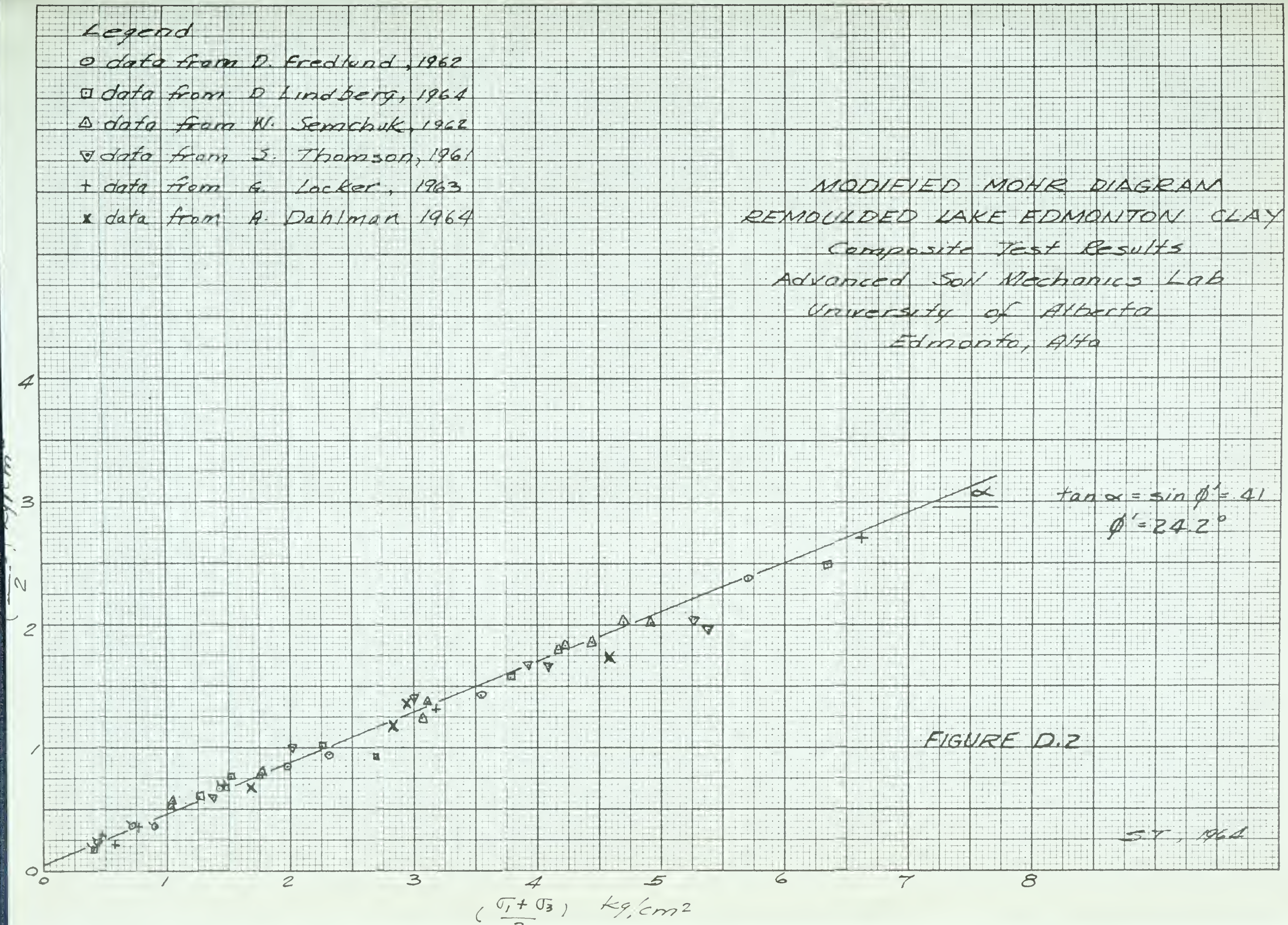

Legend
1962 normally consolidated
1962 overconsolidated
1962 Quick test
1961 Semchuk - low temp
1961 Semchuk - high temp
1964 normally consolidated
1964 overconsolidated
1960 Thomson
1961 Locker
1964 Dahlman - CFS - normally consol
1964 Dahlman - CFS - over consol.
STRENGTH TEST RESULTS
LAKE EDMONTON CLAY
Remoulded
Advanced Soil Mechanics Lab.
University of Alberta
Edmonton
S.T. 1964
Moisture Content, %
55
50
45
40
35
30
25
.1
1.0
10
Principal stress Difference, Kg/cm²

FIGURE D.1


Legend
o data from D. Fredlund, 1962
□ data from D. Lindberg, 1964
△ data from W. Semchuk, 1962
▽ data from S. Thomson, 1961
+ data from G. Locker, 1963
x data from A. Dahlman 1964
MODIFIED MOHR DIAGRAM
REMOULDED LAKE EDMONTON CLAY
Composite Test Results
Advanced Soil Mechanics Lab
University of Alberta
Edmonton, Alta
α
tan α = sin φ' = .41
φ' = 24.2°
FIGURE D.2
S.T., 1964
0
1
2
3
4
5
6
7
8
(σ1 + σ3)/2 kg/cm²